OPAC 2000: A New Pavement Design System

by

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ABSTRACT

Pavement management exists at two basic levels: network and project. The network level is concerned with determining maintenance and rehabilitation needs and developing programs for new pavement construction for various sections within overall budget constraints. The project level deals with acquiring and analysing data from those sections designated for action at the network level, carrying out the structural design and associated economic evaluation, and implementation in terms of construction and periodic maintenance and rehabilitation.

The basic objective of pavement design is to provide feasible structural alternatives with optimal service lives which minimize total life cycle costs. This is achieved by generating a series of design alternatives, performing structural and economic analyses and providing the results in an organised format, which provides the basis for the decision-making at the project level.

OPAC 2000 is a new pavement design package, which handles the pavement design process in a comprehensive computerized system. The system was developed at the University of Waterloo under a contract with the Ministry of Transportation of Ontario. This thesis provides the procedures and the background engineering principles used in the development of the system.

The following tasks were carried out. First, the existing OPAC was evaluated in light of both the requirements of a computerized pavement design system and the special needs of the system users. Second, some of the available major pavement design systems were reviewed in terms of their design methodologies, computer package availability, advantages and disadvantages. The third task was collecting pavement structure, performance history, subgrade and traffic data from in-service pavements on the Ontario highway network, from which a new set of pavement performance prediction models were established. Fourth, a new economic analysis module was developed based on the most recent Ontario and international studies. Fifth, a comprehensive system design was developed, which specified details of each design module, input and output requirements as well as the logic connections among the modules.

The key enhancements and innovations in OPAC 2000, compared to the existing OPAC system, include:

1. A new set of flexible pavement performance models,

2. Capability of carrying out overlay designs,
3. Capability of carrying out reliability analysis,
4. Capability of carrying out rigid pavement design including overlay designs, by employing the AASHTO rigid pavement design equation,
5. A new, improved and more comprehensive economic analysis module,
6. Capacity of estimating impacts to environment due to pavement works,
7. Use of the MS Windows™-based computing environment,
8. A versatile, comprehensive and "user-friendly" software package (in SI units), and
9. Demonstration of how the OPAC 2000 performance models could be used to extend the system to network level pavement management.

This thesis provides the procedures, equations and the related background engineering principles that were used in the development of the system. The following conclusions are based on the study:

1. The mechanistic-empirical nature of the OPAC pavement design method is retained in the OPAC 2000 pavement design system,
2. The OPAC pavement performance prediction model is updated based on in-service pavement performance data. Two separate models are developed based on cluster analysis: one for Southern Ontario, and one for Northern Ontario,
3. A systematic methodology was used in developing OPAC 2000 as a fully functional self-contained pavement design package,
4. A project level pavement design system should be considered within the scope of an overall pavement management system.

Although OPAC 2000 was developed for the province of Ontario, the engineering principles, the techniques and the methodology used in developing the system are believed to be transferable to other regions. Through appropriate model calibrations, OPAC 2000 type of systems could be readily adapted to such other regions.

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To my late parents,

for their care and support

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LIST OF SYMBOLS

Flexible Pavement Design

AC	asphalt concrete
H_e	equivalent granular thickness (mm)
h_i	thickness of the i^{th} layer in a pavement structure (mm)
GBE_i	Granular Base Equivalency factor of the i^{th} layer in a new pavement structure
GBE_{exi}	Granular Base Equivalency factor of the i^{th} layer in an existing pavement
m_i	drainage coefficient of the i^{th} layer in a new pavement structure
m_{exi}	drainage coefficient of the i^{th} layer in an existing pavement
W_s	Odemark subgrade deflection of the pavement structure (mm)
p	standard wheel load (kN, 40 kN on a dual tire)
M_s	modulus of the subgrade (MPa)
M_2	modulus of the granular base material (MPa, average 345 MPa)
z	$= 0.9 H_e \sqrt[3]{\frac{M_2}{M_s}}$ (mm)
a	radius of load area (mm, approximately 163 mm for an equivalent circular imprint of a dual tire)
$AADT_0$	initial year average annual daily traffic immediately after construction / rehabilitation
$AADT_j$	the j^{th} year average annual daily traffic
GR	yearly traffic growth rate
N	total number of ESAL's accumulated on the design lane after construction / rehabilitation
Y	number of in-service years of the pavement after construction / rehabilitation
T_t	total truck percentage in AADT (%)
T_i	proportion of the total truck population which belongs to truck class i (%)
TF_i	truck factor for truck class i. Either the FHWA 13 class vehicle classification scheme (omitting the first 3 classes) or the simplified four-truck class scheme can be used.
DF	direction split factor, default value = 0.5
LDF	lane distribution factor, used to account for the truck traffic in the design lane
DAYS	days per year for truck traffic. A default value of 300 is used for Ontario.
n	number of truck classes
P_T	pavement performance loss due to traffic (PCI)

Ψ	=	$3.7239 \times 10^{-6} \times W_S^{6} \times N$
P_E		pavement performance loss due to environment (PCI)
P_0		initial pavement performance index (PCI)
P_{f0}		pavement performance index after future overlays (PCI)
P_t		minimum acceptable pavement performance index (PCI)
α, β		performance model parameters
PCI		mean value predicted pavement performance index (PCI)
σ_V^2		variance due to errors in design variables
σ_M^2		variance due to errors in the regression model
σ_{Xi}		standard deviation of individual design variables
Z_R		standard normal deviate
R		design reliability
PCI_f		predicted yearly pavement performance index for given reliability (PCI)
σ_{PCI}		standard deviation of predicted pavement performance index (PCI)
h_0		future overlay thickness (mm)
GBE_0		Granular Base Equivalency factor of the future overlay material
k_i		equivalent layer reduction factors used in future overlay analysis

Rigid Pavement Design

PCC	Portland cement concrete
PSI	present serviceability index
W_{18}	accumulated number of ESAL (80 kN) applications (equivalent to N in flexible pavement design)
S_0	combined standard error of traffic and performance prediction
P_f	pavement performance index at a given year (PSI)
N_t	accumulated ESAL's corresponding to $P_f = P_t$,
D	PCC slab thickness of a design alternative, or slab thickness of the existing PCC pavement in an overlay design (in.)
D_{fo}	thickness of future PCC overlay (in.)
D_{AC}	thickness of future AC overlay (in., needs to be converted to the equivalent PCC thickness, D_{fo})
D_{sb}	subbase thickness (in.)
D_f	PCC slab thickness required by future traffic (in.)

D_{sg}	subgrade depth to rigid foundation (in.)
D_{eff}	effective thickness of the existing slab in overlay design (in.)
D_{ac}	thickness of the existing AC surface in AC/PCC pavement (in.)
D_{pcc}	thickness of the existing PCC slab in AC/PCC pavement (in.)
Sc'	modulus of rupture (psi)
Ec	elastic modulus of PCC slab (psi)
C_d	drainage coefficient
J	load transfer coefficient
E_{sb}	subbase elastic modulus (psi)
M_R	roadbed soil resilient modulus (psi)
k	modulus of subgrade reaction (psi)
k_{inf}	composite modulus of subgrade reaction (psi)
k_{MR}	composite modulus of subgrade reaction without subbase (psi)
k_{rf}	composite modulus of subgrade reaction with rigid foundation (psi)
k_{eff}	effective modulus of subgrade reaction (psi)
LS	loss of support
F_{jc}	joint and cracks adjustment factor of bonded PCC overlay
F_{jcu}	joint and cracks adjustment factor of unbonded PCC overlay
F_{dur}	durability adjustment factor
F_{fat}	fatigue adjustment factor
F_{ac}	quality factor of existing AC surface in AC/PCC pavement
N_t	accumulated ESAL's corresponding to $P_f = P_t$
N_{LS}	accumulated ESAL's corresponding to $P_f = 1.5$ and $R = 50\%$

FWD Backcalculation

FWD	falling weight deflectometer
AREA	deflection basin area (in.^{2})
d_0	maximum FWD deflection at the centre of the loading plate (in.)
d_i	FWD deflections at 30.5 cm (12 in), 61 cm (24 in), and 91.4 cm (36 in) from the plate centre (in.)
P	load on the FWD plate (lbs)
a	load plate radius (in.)
γ	Euler's constant, 0.57721566490

μ	concrete Poisson's ratio
$d_{0\,comp}$	AC compression at the center of load (in.)
E_{ac}	elastic modulus of the AC layer in AC/PCC pavement (psi)
ΔLT	deflection load transfer (%)
D_l	loaded side deflection (in.)
D_{ul}	unloaded side deflection (in.)
B	slab bending correction factor

Economic Analysis

PWTC	present worth of total cost ($/km)
PWINC	present worth of initial construction cost ($/km)
PWRHC	present worth of total rehabilitation cost ($/km)
PWMC	present worth of total maintenance cost ($/km)
PWRSC	present worth of residual cost ($/km)
PWVOC	present worth of total extra vehicle operating cost ($/km)
PWDLC	present worth of total user delay cost ($/km)
PWTMV	present worth of pavement terminal value at the end of the analysis period ($/km)
PWSLV	material salvage value at the end of analysis period ($/km)
RHC_i	rehabilitation cost at Year i ($/km)
MC_i	maintenance cost at Year i ($/km)
RHC_{last}	the last time rehabilitation cost ($/km)
PCI_t	PCI at the end of analysis period
i	number of years from the present time
r	discount rate
D_j	delay due to low speed with traffic control Plan j (hour, j = 2 to 8)
Dq_j	queuing delay with traffic control Plan j (hour)
Vr_j	reduced speed with traffic control Plan j (km/h)
Vn_j	normal speed corresponding to the reduced speed with traffic control Plan j (km/h)
HV	two-way hourly volume where Plan 4 is applied and one-way hourly volume where Plans 2, 3,5, 6, 7 and 8 are applied (vph)
$CAPr_j$	reduced capacity with traffic control Plan j (vph)

HF	hour factor, 0.125 for two-lane highways and 0.07 for other highways
D_1	average delay time under flag-person control (hour/veh)
c	reduced capacity ($CAPr_1$) under traffic control Plan 1 (vph)
X	v/c ratio, v is two-way hourly volume HV
g	green time (sec.)
C	cycle length (sec.)
DL_j	user delays with traffic control Plan j (hour)
JD_j	rehabilitation job duration (hour)
DLC_i	delay cost at Year i ($/km)
DL_i	average delays in Year i (hour)
WG	average hourly wage rate ($/hour)
$CAPn_2$	one lane normal capacity for two-lane highways (vphpl)
F_{wn}	adjustment factor for narrow lanes and restricted shoulder widths
SHD_{out}	outer shoulder width (m)
$CAPr_j$	reduced capacity using traffic control Plan j (vphpl)
$CAPn_j$	normal capacity using traffic control Plan j (vphpl)
V_p	service flow rate, the maximum volume is 2200 passenger cars per hour per lane. (pcphpl)
n	number of lanes opened to the traffic
PHF	peak hour factor, assumed to be 0.88
F_{hv}	heavy-vehicle adjustment factor for the presence of heavy vehicles in the traffic stream assumed to be 0.72
Vn_2	normal speed on two-lane highways (km/h)
Vr_2	reduced speed on two-lane highways with traffic control Plan 2 (km/h)
FFS	estimated free-flow speed (km/h)
FFSi	estimated free-flow speed under an ideal condition (60 mph)
F_m	adjustment for median type
F_{lw}	adjustment for lane width
F_{lc}	adjustment for lateral clearance
F_a	adjustment for access points, 2.5 mph assumed
SHD_{both}	lateral clearance (ft., total width of both shoulders)
Vn_3	normal speed on four-lane highways (km/h)
Vn_4	normal speed on six-lane highways (km/h)
IRI	International Roughness Index (m/km)

VOCAIRI	extra VOC due to deterioration of IRI for vehicle group A ($/1000 veh-km)
VOCBIRI	extra VOC due to deterioration of IRI for vehicle group B ($/1000 veh-km)
VOCCIRI	extra VOC due to deterioration of IRI for vehicle group C ($/1000 veh-km)
$VOCA_i$	VOC for Group A at Year i ($/km)
$VOCB_i$	VOC for Group B at Year i ($/km)
$VOCC_i$	VOC for Group C at Year i ($/km)
TYA	proportion of vehicle group A in AADT
TYB	proportion of vehicle group B in AADT
TYC	proportion of vehicle group C in AADT
$PWVOC_i$	present worth of extra VOC at Year i ($/km)
COA, COB, COC	carbon monoxide emission from vehicle groups A, B and C, respectively (kg/1000 veh-km)
HCA, HCB, HCC	hydrocarbon emission from vehicle groups A, B and C, respectively (kg/1000 veh-km)

CHAPTER 1 INTRODUCTION

1.1 Background

A major problem facing engineers and researchers in the pavement community is the complexity in pavement design, which in turn impacts the decision-making of pavement management at both the project level and the network level. The properties of materials used in construction, the traffic loads carried by the pavement, and the environment in which the pavement functions all have a role in the pavement's performance. Furthermore, many of the factors that need to be considered in the pavement design process are "dynamic" in nature. For example, the paving materials used nowadays, the construction methods and equipment as well as the characteristics of traffic loads are quite different from those of decades ago. In addition, pavement designers have to consider the cost issue in selecting pavement design alternatives as it often represents a major part of the total investment in building and maintaining a highway network.

There have been many computerized pavement design systems developed to help engineers, along with their experience and engineering judgments, in designing pavements. As an example, the Ontario Pavement Analysis of Costs (OPAC) system was developed by the Ministry of Transportation of Ontario (MTO) in the mid 1970's [Jung 1975, Kher 1975] and has been used as the principal pavement design tool ever since [MTC 1980].

During the past two decades, however, pavement design needs in Ontario experienced substantial changes. Due to the limitations in the performance models, lack of comprehensiveness, and lack of versatility of the existing OPAC, MTO initiated a project in 1992 to update OPAC and the contract was awarded to the University of Waterloo (Project No. 21184).

OPAC 2000 is the name of the project—it incorporates both new engineering and economic analysis procedures and a comprehensive computer software package. The project was planned to be carried out in four stages: Stage 1 involved identifying functional and operating requirements of the new system, as summarized in the writer's M.A.Sc. thesis [He 1993]. Tasks in Stage 2 included developing pavement performance models and detailed computer software system design. Stage 3 was development of the software package. Stage 4, subsequent to this thesis, is planned for training sessions and on going system support.

This thesis describes the engineering and economic analysis procedures and the system design of the OPAC 2000 computer software package. Working examples are also provided in various parts of thesis to explain the analysis procedures.

1.2 Objectives and Scope of the Research

While the main objective of the research was to develop a new comprehensive, practical pavement design system for Ontario, as subsequently described in Chapter 2, accomplishment of the following tasks was a key to complete the project.

First, the existing OPAC was evaluated in light of both the requirements of a computerized pavement design system and the special needs of the design system users. This task was finished in the early stage of the project through investigating the existing OPAC system and a series of interviews with MTO regional pavement design engineers [He 1993].

Second, some of the available major pavement design systems were reviewed in terms of their design methodologies, computer package availability, advantages and disadvantages. This task was initiated in the first stage of the project, and expanded later on as some newer pavement design systems evolved.

The third task was collecting pavement structure, performance history, subgrade and traffic data from in-service pavements on the Ontario highway network, from which to establish a new set of pavement performance prediction models. These models form the backbone of the flexible pavement design modules in OPAC 2000.

Fourth, a new economic analysis subsystem was developed based on the most recent Ontario and international studies. As an enhancement, a vehicle emission prediction subroutine is included in the OPAC 2000 package to meet the requirements of assessing environmental impacts of pavement rehabilitation.

Last, but not least, a comprehensive system design was carried out which specified details of each design module, input and output requirements as well as the logical connections among the modules. The system design was used to guide the development of the software package and the OPAC 2000 user's manual.

1.3 Organization of the Thesis

This thesis consists of nine chapters. Chapter 2 generally describes the requirements of the new OPAC 2000 pavement design system. A brief review of the existing OPAC system is also included.

Chapter 3 covers the findings from the review of available pavement design systems, which included some new developments. The structure of the OPAC 2000 pavement design system was developed based on the findings.

Chapter 4 summarizes the design procedure for flexible pavements. A major part in this Chapter is the calibration of the pavement performance prediction model. As well, a reliability analysis procedure was developed and is explained in detail.

Chapter 5 provides the framework and the procedure for rigid (portland cement concrete, PCC) pavement designs based on the 1993 AASHTO Guide for Structural Design of Highway Pavements [AASHTO 1993] (referred to as the AASHTO Guide hereafter).

The economic evaluation module is one of the major subsystems of OPAC 2000. The life-cycle cost analysis procedure in this module is presented in Chapter 6. It involves calculating both the agency costs and road user costs. A sub-module for estimating vehicle emission effects associated with pavement rehabilitation activities is included.

Chapter 7 describes the development of the software system structure. A major feature of the user interface is the report module which consists of the design input/output report, the sensitivity analysis report and the graphics, etc.

Chapter 8 provides the outlook and an example of how the OPAC 2000 pavement performance models can be used in the network level of pavement management.

Finally a summary of the newly developed pavement design system and recommendations toward future improvements are covered in Chapter 9.

CHAPTER 2 REQUIREMENT OF A PAVEMENT DESIGN SYSTEM

Pavement management systems have seen rapid development in both the theory and the practice in the past two decades. Pavement design is an essential component in the overall pavement management process. At the project level, it arrives at a structural design, determines the initial investment, and the future maintenance and rehabilitation needs from the budget of the agency. As well, it influences the traveling public in the form of the road user costs. For example, poor pavement conditions resulting from inadequate pavement design or insufficient funding for maintenance and rehabilitation will cause an increase in vehicle operating costs to road users. Therefore, appropriate pavement design decisions need to be based on sound technical and economic analyses.

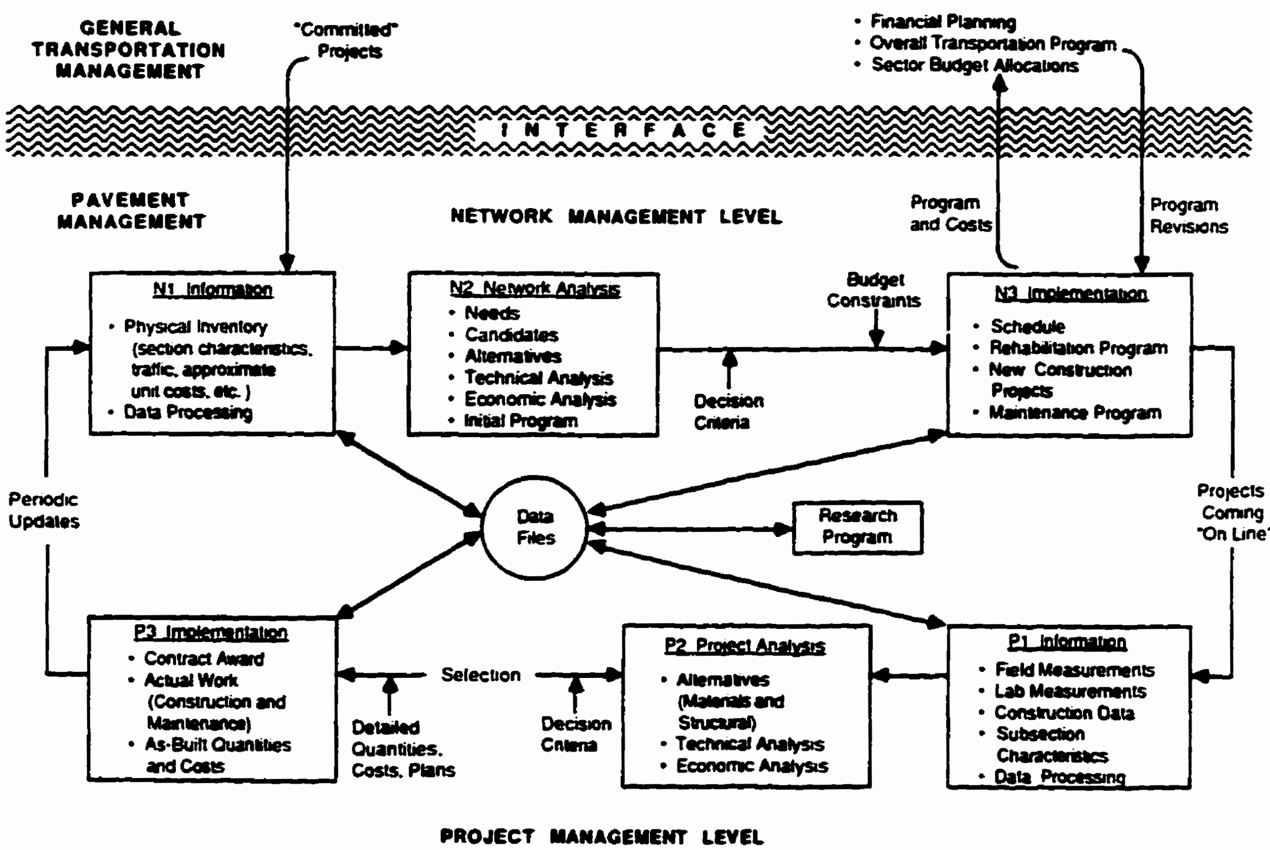

Figure 2. 1 Information Flows in a Pavement Management System [Hudson 1979]

The network level of pavement management involves data acquisition, "needs" analysis, rehabilitation alternative analysis and network strategy optimization analysis. The often-constrained budget is then allocated to the road network to achieve the highest cost-effectiveness. The basis of the analyses is the project level pavement data including the structural data, traffic data and pavement performance data. Figure 2.1 shows schematically the data flow between two levels of pavement management.

In pavement management systems, the design activity is not a stand-alone task for an individual pavement section, but part of a series of systematic activities. To ensure data used in and provided by the pavement design process meet the requirement of future analysis at both the project and network levels, the pavement design itself needs to be considered as a subsystem in the overall pavement management system.

2.1 Pavement Design as a Subsystem

The basic objective of pavement design is to provide structural alternatives that are feasible both technically and economically. This is achieved by specifying pavement layer thicknesses with proper types of materials based on the traffic and environmental conditions and by life-cycle cost analysis to the project being designed. Figure 2.2 shows a framework of a pavement design subsystem [Haas 1994]:

While there are different ways of achieving the objective of pavement design, Figure 2.2 shows generally that in designing a pavement, three major groups of activities need to be conducted: (1) Input information relating to materials, traffic, climate and costs plus selection of a design period, structural and economic models, the identification of objectives and constraints, and variance data on the inputs to the model. (2) Generating design alternatives with specified design strategies (i.e. material types and thicknesses, criteria on structural and economic analysis, etc.). (3) Structural analysis and economic evaluation of the alternatives and the process to select the best strategy for implementation. The activities involved in the pavement design process include the input and output data, the separate analysis modules as well as the report generating module. All these need to be organised with a systematic methodology.

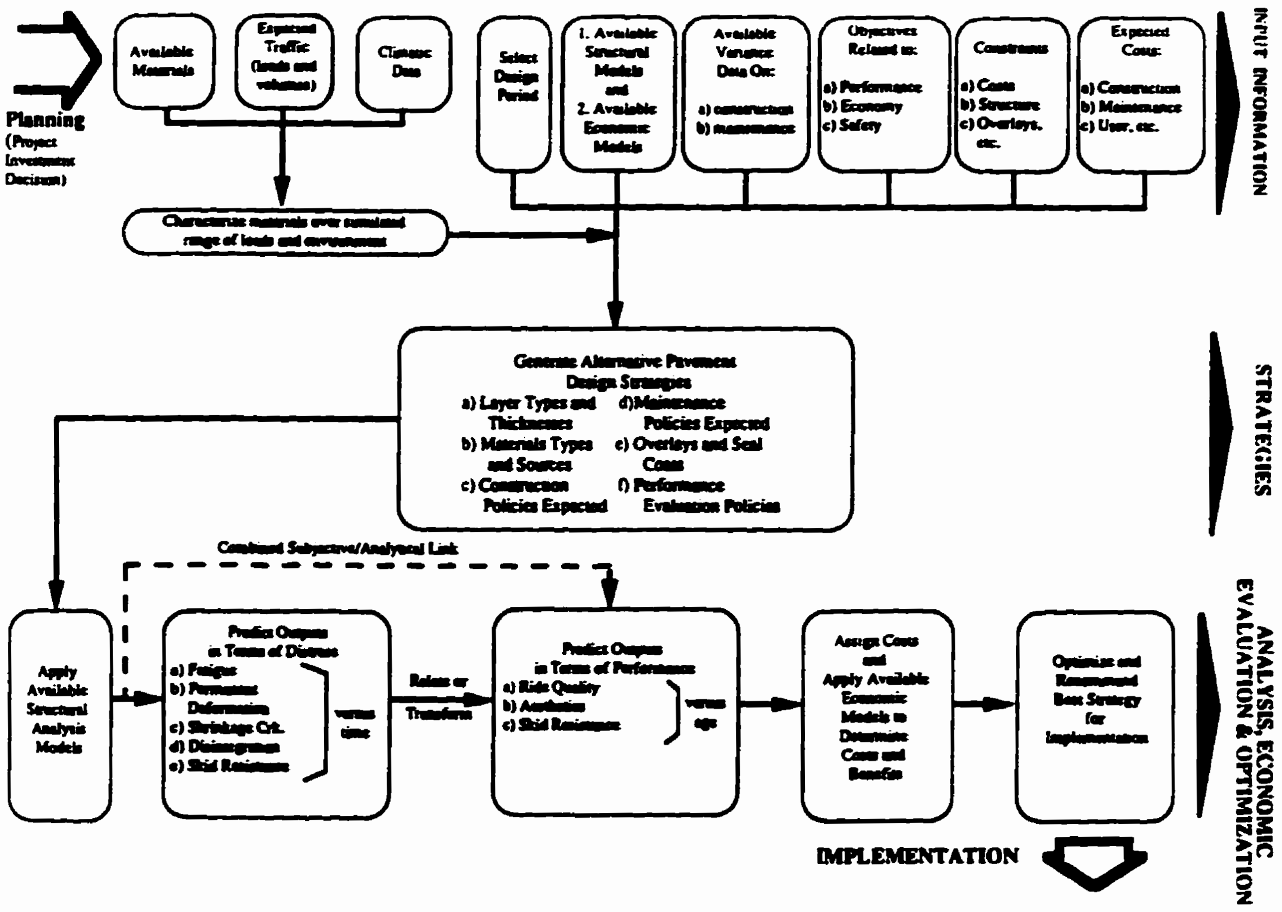

Figure 2. 2 Framework of Pavement Design [Haas 1994]

2.2 Identifying Requirements of the New Pavement Design System

To achieve the goal of providing technically and economically feasible design alternatives, a satisfactory pavement design needs to address the following issues [He 1993]:

1. Pavement type, i.e., flexible, rigid or composite pavements,
2. Requirements of material properties,
3. Design criteria, such as performance levels, life span, reliability, etc.,
4. Maintenance and rehabilitation policies,
5. Economic analysis of various pavement design alternatives, and

6. Recommendation of the optimal design strategies.

With these general requirements in mind, an appraisal of the existing OPAC system was carried out. It included a careful review of the current PC version of the software (OPAC2M). A successful pavement design system has to be able to address the local needs of the highway agency; more specifically, the potential users of the system. In addition to the new system development, as substantially described, significant efforts went into a comprehensive user survey of the MTO head office and regional pavement design engineers. The detailed survey results are summarized in a previous study [He 1993], with the major findings restated as follows.

The appraisal revealed the following positive characteristics of the existing OPAC system:

1. Separation of the performance model into traffic and environment associated loss components,

2. Simplification of the actual pavement structure into an equivalent two-layer structure for structural analysis,

3. Capability of using fundamental or mechanistically based properties of the pavement layer materials (i.e., moduli),

4. Capability of calculating a fundamental response of the pavement (i.e., deflection)

5. A comprehensive economic analysis model which converts all costs to present worth, and which incorporates user costs.

It was also determined that the existing OPAC system has some key limitations in fulfilling the requirements of various pavement design tasks [He 1993]:

1. OPAC can be used only for new flexible pavement designs, not overlay designs for pavement rehabilitation projects which represent the majority of current pavement design tasks,

2. The existing OPAC system does not have the capability of designing portland cement concrete (rigid) pavements and associated rigid pavement overlays,

3. The pavement performance prediction model for environment associated deterioration is based on performance data of only 8 years, and the traffic associated part of the model is questionable when the number of Equivalent Single Axle Load (ESAL) applications

reaches the level of more than 7 million [Jung 1992]. These models are not linked with reliability measurements for the performance predictions,

4. The user cost model is based on vehicle operating cost (VOC) relationships from the mid 1970's, and it is not clear from both the existing OPAC documentation and the system output whether the user cost item listed for pavement design alternatives contains both vehicle operating cost (VOC) and the traffic delay cost,

5. The computing environment of the existing OPAC is not "user-friendly". It requires a time consuming sequential set of steps for operation, and has little flexibility. The system does not have graphic presentation capabilities (i.e., plotting pavement performance predictions, economic analysis results, etc.). It is realized that the user-friendliness problem is largely attributable to limitations of computer technology at the time the OPAC system was developed.

The software appraisal and the survey results also showed that despite the shortcomings, the basic principles used in the structural and economic analysis modules are still valid. The foregoing weaknesses, however, can be considered as requirements for the new system OPAC 2000.

CHAPTER 3 DEVELOPMENT OF THE STRUCTURE OF OPAC 2000

This chapter describes the general organization of OPAC 2000. As the initial step of designing the new pavement design system, the basic building blocks or modules are identified and a framework of the system is developed. Based on the modules and the framework the system design of the software package is performed. To support these efforts, the basic types of pavement design methods and the available pavement design systems were reviewed as summarized in the chapter.

3.1 Categories of Pavement Design Methods

There are a variety of pavement design methods used by different highway agencies in different times, and the way of categorizing pavement design methods varies from author to author. Haas et al divided pavement design methods chronologically into 6 classes [Haas 1994]:

1. Methods based on experience (1920-1930's), where a certain pavement structure is linked with a standard service life without excessive distresses. This method can be relatively reliable for particular jurisdictions,
2. Methods based on "Soil Formula" (1930's): the methods are based on simple soil classification tests and empirical correlations with pavement thickness,
3. Methods based on simple strength tests (1930's), where simple procedures of measuring material properties are established and the material properties are related with pavement thickness,
4. Methods based on field or laboratory strength tests (1940's): field or laboratory tests are performed to obtain pavement layer and subgrade moduli and the moduli are used in theoretical analysis procedures in order to limit deflection or to ensure stability,
5. Methods based on the elastic layered theory (ELT, 1950's): pavement thickness is determined by considering distress mechanisms such that certain critical stresses, strains or deflections are not exceeded, and

6. Methods based on statistical evaluation of pavement performance (1960's), where pavement thicknesses are determined based on performance prediction and economic comparison of alternatives.

With the evolution of computer technology, recent developments in pavement design focused more on the latter two types of methods. Depending on the way the designed structure is evaluated in a computerized pavement design system, Rauhut et al classified pavement design methods into three basic types [Rauhut 1987]:

1. Design based on empirical pavement performance models, where pavement performance is evaluated with a mathematical relationship developed from field data,

2. Design based on mechanistic analysis: design alternatives are evaluated through analyzing mechanistic response of the pavement structure, such as stress, strain and deflection, and

3. Design based on mechanistic-empirical performance model: pavement performance model is developed by employing mechanistic models combined with field data.

The design methods based on empirical pavement performance models can be reliable for the jurisdictions for which they are developed. A key limitation of empirical methods is that they are hard to use in the regions where the field conditions are different from those used in developing the methods. One researcher reported that the pavement performance models of some empirical methods based on road tests may not adequately predict field performance of in-service pavements even within the inference space of the models [Daleiden 1994].

Mechanistic methods are based on analysis of the primary responses in the pavement structure, such as strain, stress and deflection. Two factors contribute to the limited use of mechanistic models in highway agencies: (1) mechanistic methods typically require inputs from extensive laboratory testing and relatively precise field measurements, and this is not always practical to highway agencies; (2) researchers in this field have realized that pavement performance will likely be influenced by a number of factors which will not be precisely modeled by mechanistic methods [AASHTO 1993].

The mechanistic-empirical approach is getting increased attention in various highway agencies and research bodies. The procedure is to calibrate the mechanistic (primary response) performance prediction model with observed performance indices, i.e., empirical correlation [Paterson 1992]. Main benefits of such a procedure include: (1) improved reliability, (2) ability to predict specific types of distresses, (3) ability to extrapolate from limited field and laboratory results [AASHTO 1993]. This type of model was recommended for the recent SHRP/LTPP studies in the United States and Canada [Rauhut 1987]. It can be seen in a later section that the OPAC pavement design system is also of this type.

3.2 Evaluation of Available Pavement Design Methods and Computer Software

In order to gain insight for the OPAC 2000 project, some of the available pavement design systems were evaluated in terms of the analysis methodology, advantages and disadvantages. A second purpose of the assessment was to look for a candidate rigid pavement method for the OPAC 2000 system as an Ontario based rigid pavement method does not exist. It was not intended to have a complete list for the pavement design systems developed in the past, but the systems chosen were those being used extensively both in North America and in other places in the world. These included the AASHTO flexible and rigid pavement design methods, the Asphalt Institute method, the U.S. Federal Highway Administration (FHWA) method, the Shell method, and the Portland Cement Association (PCA) method.

Discussions of these design systems and their associated computer programs were documented in an earlier study [He 1993]. Later in 1993, two more pavement design system were made available: the DARWin™ Pavement Design System developed by ERES Consultants, Inc. [ERES 1993] and the PAS system by American Concrete Pavement Association [ACPA 1993], both based on the 1993 AASHTO "Guide for the Design of Pavement Structures"[AASHTO 1993]. Table 3.1 summarizes the findings of the study, with updates on the two software packages: DARWin™ and PAS.

The pavement design methods mentioned in Table 3.1 can be categorised as empirical (AASHTO) or mechanistic (all others). The AASHTO method can be used for flexible, rigid pavement and overlay designs, while others work only on flexible or rigid pavement design. Some of the methods do not provide an overlay design procedure. The use of pavement performance concept (PSI), the load equivalence factors (LEF's) and the equivalent single axle load (ESAL) calculation procedure makes the AASHTO method more practical to pavement engineers. For these reasons the

AASHTO rigid pavement design method is selected as the basis of the OPAC 2000 rigid pavement design module.

Table 3.1 Evaluation of Different Pavement Design Systems

DESIGN METHOD/ SYSTEM	STRENGTH	WEAKNESS
AASHTO (DARWin™ and PAS)	1. Design procedure available for flexible, rigid pavements and overlay designs 2. Rigorous or simplified ESAL calculation 3. Present serviceability index (PSI) concept developed to evaluate pavement performance 4. Drainage condition can be considered 5. Available life cycle cost (LCC) analysis 6. Available sensitivity analysis procedure 7. Available backcalculation of FWD[1] measures	1. Questionable performance prediction for flexible pavements 2. LCC analysis not linked with structural analysis
Asphalt Institute (HWY, LCCost)	1. Design procedure available for flexible pavements and overlay designs, including emulsified asphalt material 2. ESAL calculation routine based on AASHTO 3. Guidelines for selecting asphalt grade and modulus of asphalt mix under different climate conditions 4. Provision for staged construction 5. Available life cycle cost (LCC) analysis	1. Not linked with pavement performance 2. LCC analysis not linked with structural analysis
FHWA (VESYS)	1. Design procedure available for new flexible pavements 2. Use of stochastic inputs and output distributions of pavement performance (PSI), and distresses (cracking and rutting) 3. Adaptability for a broad range of traffic and climatic conditions	1. Complicated procedure; impractical for routine uses 2. Lack of rigid pavement design, pavement overlay design and LCC analysis procedures

[1] Pavement deflection measured by the Falling Weight Deflectometer (FWD) as a strength parameter. Backcalculations can be performed based on the FWD measures to estimate moduli of both the pavement materials and the subgragde.

Table 3.1 Continued

DESIGN METHOD/ SYSTEM	STRENGTH	WEAKNESS
SHELL (Computer package)	1. Design procedure available for new flexible pavements and overlays 2. Considers climatic impacts 3. Available rut depth prediction subroutine	1. Not linked with pavement performance 2. Lack of cost analysis procedure
PCA (PCAPAV)	1. Applicable to all types of portland cement concrete pavement (PCC) designs (plain, reinforced, continuous reinforced) 2. Estimates slab fatigue and subgrade erosion 3. Considers effects of concrete shoulder, curb or gutter, etc.	1. Not linked with pavement performance 2. Lack of cost analysis procedure

3.3 Development of the Structure of OPAC 2000 Pavement Design System

A careful evaluation of the existing OPAC pavement design method identified its strengths and weakness and concluded that the basic engineering principles and methodology used are still valid. The study has set the stage for developing the structure of the new OPAC 2000 pavement design system. It was considered essential in this project to keep the strengths of the existing OPAC while making improvements to the shortcomings. The major focus of developing the new system is on the following areas:

1. Calibrating the flexible pavement performance prediction model with MTO in-service pavement performance data,
2. Developing OPAC 2000 rigid pavement design module based on the AASHTO rigid pavement design method,
3. Expanding design pavement types to include overlay designs for both flexible and rigid pavements,
4. Incorporating a reliability concept in the structural analysis modules,

5. Developing a comprehensive economic analysis module with new agency cost data and vehicle operating cost study results, the addition of a new user delay cost model and a new emission effect model,

6. Developing a versatile computing package based on a complete system design for a Microsoft® Windows environment

Based on previously identified requirements, a general structure of the new system OPAC 2000 is given in Figure 3.1. It basically illustrates that the two major blocks, structural analysis and economic analysis, receive input from data and command files, that data and graphic outputs can be included in the reports generated and that the user is provided with on-line help.

3.3.1 Structural Analysis Modules

There are seven structural analysis modules in OPAC 2000 for performing various types of pavement designs: (1) New flexible (AC) pavement analysis module, (2) New rigid (PCC) pavement analysis module, (3) AC overlay of AC pavement analysis module, (4) AC overlay of PCC pavement analysis module, (5) AC overlay of existing AC/PCC pavement analysis module, (6) Bonded PCC overlay of PCC pavement analysis module, (7) Unbonded PCC overlay of PCC pavement analysis module.

The flexible pavement design and flexible overlay design modules (No. 1 and 3) are based on the new OPAC 2000 pavement performance prediction model as developed in Chapter 4. The rigid pavement design and rigid overlay design modules (No. 2, 4, 5, 6 and 7) are based on the AASHTO Guide [AASHTO 1993] as described in Chapter 5. These modules are included in the block of STRUCTURAL ANALYSIS.

The output from the structural analysis blocks become the input to the economic evaluation module for the life-cycle cost analysis, which includes the predicted pavement performance, material types, layer thicknesses, lane width and number of lanes and the shoulder information.

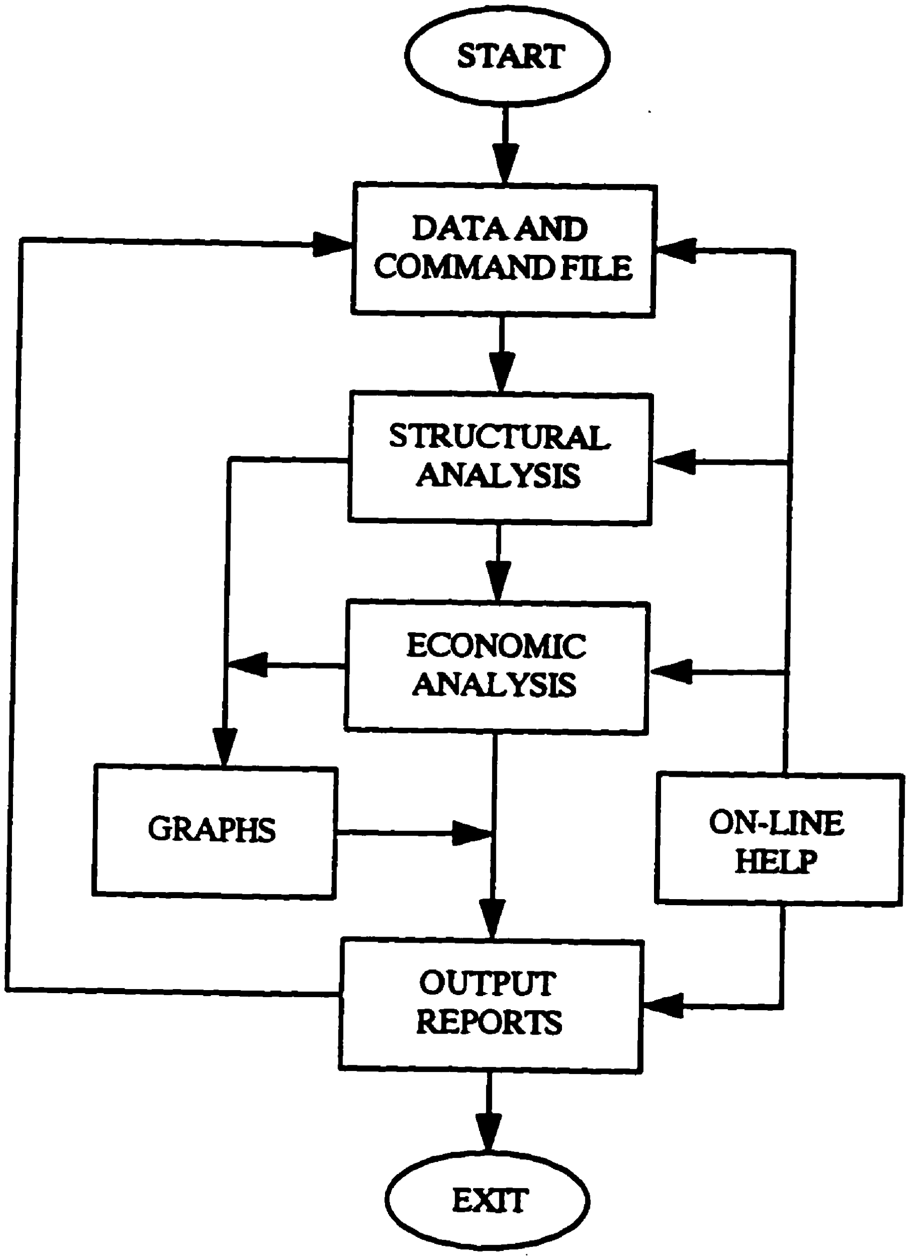

Figure 3. 1 OPAC 2000 System Overview

3.3.2 Economic Analysis Module

This module takes the pavement performance data and structural design data generated from the structural analysis modules to calculate life-cycle costs of pavement design alternatives. Results of the economic analysis are the present worth of the agency costs and road user costs. Agency costs include the initial construction cost, maintenance cost, rehabilitation cost and residual value. Road user costs include vehicle operating cost and user delay cost due to pavement rehabilitation. The total cost of each pavement design alternative is calculated by adding up agency costs and user costs and subtracting the residual value in terms of the present worth. Detailed description of the cost analysis procedures is provided in Chapter 6.

As an option, a vehicle emission prediction output related to pavement rehabilitation activities is also made available from the economic analysis module because of sharing the same vehicle speed calculation procedure. The model predicts the amount of two major pollutants: carbon monoxide (CO) and hydrocarbon (HC). The inclusion of this model will meet the potential requirement that the planned project is facing environmental assessments.

3.3.3 Other Modules

The remainder of Figure 3.1 includes the blocks of DATA AND COMMAND FILE, GRAPHS, OUTPUT REPORTS and ON-LINE HELP. These are dealt with by the user interface in the software package.

The DATA AND COMMAND FILE block allows the user to give design inputs and select an appropriate module to perform analyses based on the inputs. In the software package the result of the analyses can be presented both in figures and in organized tables through the GRAPHS and OUTPUT REPORTS blocks. The ON-LINE HELP block provides messages assisting the user in manipulating the system. A material library and a maintenance activity library are also designed to be available "on line" to facilitate input operations. More detailed information on this part is provided in Chapter 7.

CHAPTER 4 FLEXIBLE PAVEMENT DESIGN MODULES

Both the new flexible pavement design module and the flexible pavement overlay design module in OPAC 2000 are based on a set of newly developed pavement performance prediction models which incorporate reliability analysis. This chapter describes the effort made to calibrate the OPAC pavement performance prediction model (Section 4.1) and the organization of the OPAC 2000 flexible pavement design modules. The reliability analysis method and the design procedures of flexible pavement design modules are documented in Sections 4.2 and 4.3, respectively. A sample analysis is given in Section 4.4.

4.1 The New Pavement Performance Prediction Model

Through the descriptions in this chapter and the following chapter it can be found that the pavement performance prediction model is the basis of the OPAC 2000 pavement design system. The development of the new pavement design system started with working on the pavement performance prediction models. The existing performance prediction model and the structural analysis procedure is first reviewed in this section. It is followed by the efforts of in-service pavement performance data collection and processing and the method used in developing the new models.

4.1.1 Performance Model and Structural Analysis Procedure in the Existing OPAC

The pavement performance prediction model in the current OPAC was developed based on the AASHO and the Brampton road tests. The model is divided into two parts: the traffic-related part and the environment-related part, as expressed by the following equation:

$$P = P_0 - P_T - P_E \tag{4.1}$$

where: P is the predicted pavement performance, P_0 is the initial pavement performance index, and P_T and P_E are the performance losses due to traffic and environment, respectively. At the time the model was developed, RCI (Riding Comfort Index) was used as the pavement performance index in Ontario. RCI is expressed on a scale of 0 to 10, with 10 being the condition of newly constructed pavements and 0 being the worst condition. The basic equations in the OPAC performance prediction model are as follows:

$$H_e = a_1h_1 + a_2h_2 + a_3h_3 \tag{4.2}$$

where: H_e (mm) is the equivalent granular thickness. h_1, h_2 and h_3 are the actual thicknesses of the asphalt concrete, granular base and subbase layers. a_1, a_2 and a_3 are the strength coefficients of the asphalt concrete, granular base and subbase layer materials, which are also called "granular base equivalency (GBE) factors".

This calculation of equivalent granular thickness allows the pavement to be transformed from a multi-layered system into a two-layer equivalent structure, and thus the (Odemark) subgrade deflection, W_s, can be calculated as [Jung 1975]:

$$W_s = 1000 \times \frac{p}{2M_sZ\sqrt{1+\left(\frac{a}{Z}\right)^2}} \tag{4.3}$$

where:

p	=	standard wheel load (i.e., 40 kN on a dual tire)
M_2	=	modulus of the equivalent granular base material (the average value is 345 MPa)
M_s	=	modulus of the subgrade (MPa)
Z	=	$0.9H_e\sqrt[3]{\frac{M_2}{M_s}}$
a	=	radius of loaded area (i.e., approximately 163 mm for an equivalent circular imprint of a dual tire).

The calculation of the Riding Comfort Index losses due to traffic, P_T or ΔRCI_T, is as follows[2]:

$$\Delta RCI_T = 2.4455\Psi + 8.805\Psi^3 \tag{4.4}$$

where:

Ψ = $3.7239 \times 10^{-6} \times W_S^6 \times N$ (for W_S in mm)

[2] The equation was derived from a study of the relationship between the Odemark subgrade deflection and the pavement performance data from the AASHO road test ($R^2 = 0.9$) [Kher 1977].

N	=	number of (80 kN or 18 Kip) Equivalent Single Axle Load (ESAL) applications

This equation was obtained from regression analysis of the AASHO Road Test results [Jung 1974].

The RCI losses due to environment, P_E, is expressed as:

$$P_E = (P_0 - P_\infty)(1 - e^{\alpha Y}) \tag{4.5}$$

where:

α	=	constant
Y	=	pavement age
P_0	=	initial performance
P_∞	=	performance at an infinite time

Equation (4.5) shows that for a particular pavement section the maximum amount of environment induced performance loss is determined by ($P_0 - P_\infty$), and the rate of loss is at a maximum in the initial years and reduces with time as P_E approaches a hypothetical ultimate value of P_∞ at infinite time. The asymptotic value of P_∞ of a pavement can be made a function of W_S:

$$P_\infty = \frac{A}{1+\beta W_s} \tag{4.6}$$

where: A and β are constants.

Since P_∞ is larger for stronger pavements (small W_S), it can be found that, by substituting P_∞ into Equation (4.5), P_E is smaller for stronger pavements. Therefore, Equation (4.5) indicates that stronger pavements will be less affected by environmental forces as compared with weaker pavements [Jung 1975].

The Ontario Brampton Road Test was used to determine the constants in the above P_E model [Phang 1981]. The final equation for calculating the environment-associated performance loss, P_E or ΔRCI_E, in the OPAC model is given as[3]:

$$\Delta RCI_E = (RCI_o - \frac{A}{1+\beta W_S})(1 - e^{\alpha Y}) \tag{4.7}$$

where:

[3] The R^2 and the Standard Error of Estimate (SEE) of the equation are not provided with the literature reviewed.

RCI_0 = initial RCI

W_S, Y = as previously defined.

In MTO's pavement management database, pavement performance is presently measured in terms of Pavement Condition Index (PCI). It takes into account both riding quality and surface distresses by the following empirical relationship [MTO 1990]:

$$PCI = 100(0.1RCR)^{\frac{1}{2}} \frac{205 - DMI}{205} C + S \qquad (4.8)$$

where: RCR is the riding quality measured by the Portable Universal Roughness Device (PURD), and DMI is the Distress Manifestation Index, a weighted sum of the amount and severity of fifteen individual pavement distresses such as rutting, rippling, various types of crackings, etc. Constants C and S are equal to 1.077 and zero in this relationship, respectively [MTO 1990]. PCI is on a scale of 0 to 100. It is approximately ten times greater in numerical value than RCI in the original models.

4.1.2 Strategy of Updating the OPAC Model

The method of separating performance loss due to traffic and environment is a unique feature of the foregoing formulation[4]. This strategy is endorsed by the AASHTO Guide for Design of Pavement Structures [AASHTO 1993]. The concept is given as:

$$\Delta PSI = PSI_{traffic} + PSI_{Swell/Frost\ Heave} \qquad (4.9)$$

This is very similar in approach to the OPAC model. The final shape of the performance curve is determined by the combination of traffic and environmental effects.

Selecting a proper mathematical form is a important step for building a performance model. With regard to updating the OPAC model, there were two key considerations: (1) it was considered important to retain the capability of separately modelling P_T and P_E, and (2) the mathematical form of the existing OPAC model has good engineering significance and hence should be maintained.

The traffic-related part (P_T term) in the OPAC model is based on the AASHO Road Test in which the accelerated traffic loading was the dominant factor of performance loss. For this project it is considered that this part should be retained until newer study results with load-intensive test data are made available.

[4] It is realized that a clear separation between the traffic and the environment is very hard to achieve in practice; any interaction effects are shared by the two terms (P_T and P_E).

The environment-related part (P_E term) in the model was from the Brampton Road Test of which performance data was available for only eight years. Due to the fact that only one geographic location was used in the Brampton Road Test, together with a short period of performance monitoring, the calibration or updating of the pavement performance model is focused on the P_E part, i.e., Equation (4.7), in this project.

The existing OPAC model (Equations 4.1, 4.4 and 4.7), with the coefficients determined at the Brampton Road Test, has been used as a "common model" in Ontario for the past 20 years. Because of the widespread nature of the highway network in Ontario, it can be shown that to fit such a common model to pavement conditions in the entire province is very difficult. In order to reduce the overall deviation of prediction, the model updating strategy included the following steps:

1. Data collection and processing, including clarifying traffic, structure and subgrade soil information, checking for possible unreasonable observations,
2. Classifying: subdivide pavement sections in the database into smaller groups by employing cluster analysis and engineering judgements,
3. Retain the existing traffic loss part of the model and calibrate the environmental part of the model, specifically, coefficients α and β, and
4. Model verification.

4.1.3 Data Collection and Processing

A significant amount of effort was devoted into acquiring the necessary data base and developing the new flexible pavement performance prediction model.

The existing OPAC pavement performance prediction model was based on a limited database from 36 test sections with 8 years of performance observations [Phang 1981]. Because of the inherent variability of pavement material properties and the lack of precise measuring of traffic load and environmental effects, performance models built on road tests need to be calibrated in an iterative process based on long-term pavement performance data [Hicks 1987]. Acquisition of long-term pavement performance data is crucial to the model calibration in the project. Thus, considerable work occurred in acquiring the data for building the database.

With the help of MTO staff in both the head office and the regions, performance data from more than 100 pavement sections were collected, among which 94 sections from all over the province are considered relatively complete and hence are used for model calibration[5]. Table 4.1 lists the distribution of the sections. Appendix A contains a sample of the collected pavement performance data sheets. A complete listing of the 94 sections can be found in Appendix B, in which the "Begin Year" and the "End Year" mark the period for which pavement performance data were available.

Table 4.1 Distribution of Pavement Sections Used for Model Calibration

Region	District (#)	No. of Sections
South West	Chatam (1)	2
	London (2)	4
	Stratford (3)	18
	Owen Sound (5)	4
Central	Burlington (4)	3
	Toronto (6)	14
Eastern	Port Hope (7)	3
	Ottawa (9)	19
	Bancroft (10)	10
Northern	Huntsville (11)	10
	New Liskeard (14)	3
Northwestern	Sault Ste. Marie (18)	2
	Thunder Bay (19)	2
Sum		94

4.1.3.1 Structural Data Processing

Structural data includes the layer thicknesses and moduli of the pavement materials and the subgrade. They are carried on the "Action Plan Fact Sheets" from the MTO pavement management

[5] MTO has approximately 3,000 sections in its pavement management database. However, complete data has not yet been collected for most of the sections, which is necessary for long term performance modeling.

database. Some of the missing data on the sheets were acquired by visiting the regional pavement engineers. It should be mentioned that pavement coring tests would be valuable to acquire more accurate structural data, but it was decided not to perform the tests due to the large amount of potential test sections and the constraints of time and resources.

The structural data are used in calculating the Odemark subgrade deflection, i.e., "W_s" in Equation (4.3), which links thickness with pavement performance. Standard GBE factors are used in calculating the equivalent thickness of pavement structures using Equation (4.2), i.e., using 2.0, 1.0, 0.67 for asphalt layer, granular base and subbase, respectively. For structures with asphalt overlays an $a_1 = 2.0$ is used for the new material, and an $a_2 = 1.25$ is used for the old asphalt material. The GBE values (a_i's) are determined based on Table 3.5 of the MTO "Pavement Design and Rehabilitation Manual" [MTO 1990].

Average modulus values (M_s) are used for various types of subgrade. The M_s values are listed in Figure 3.13 of [MTO 1990]. Converted modulus values in the SI units (MPa) are subsequently provided in Section 4.3.

4.1.3.2 Traffic Data Processing

The OPAC model requires estimating the number of traffic loads to be carried by the pavement in terms of the standard 80 kN equivalent single axle load, i.e., ESAL's. The accumulated ESAL number is used in Equation (4.4) as the N value to estimate the traffic associated pavement performance loss. The current method uses the annual average daily traffic (AADT), truck percent (Truck %) and the heavy commercial truck percent (HCT%) data to determine the truck factor (TF) in calculating the N value. TF represents the number of ESAL's per truck. Because the HCT% data is often not readily available for use in pavement designs, a new method was used to estimate the truck factor (TF) for various highway classes, which is based on a recent study of Ontario highway traffic loading [Hajek 1995a, 1995b]. The truck factors used in calculating the N value for the model calibration are given in Table 4.2.

The truck factors shown in the table are for all truck classes for the corresponding road class, because the truck class distribution is not available in the historical traffic data. They are used as the default values. Typical truck factors for different truck class, representing the current situation, are provided in Tables 4.6 and 4.7.

Table 4.2 Truck Factors for Different Road Classes

Road Class	Collector	Minor Arterial	Principal Arterial
Urban	0.61	1.11	1.63
Rural	0.65	1.47	2.02

4.1.4 Pavement Section Grouping with Cluster Analysis

As pointed out earlier it is very difficult to predict performance for the pavements in the whole province using a single model, such as the existing OPAC model. Cluster analysis was therefore chosen as a means to subdivide pavement sections in the pavement performance database into smaller highway networks in order to reduce the overall prediction error of the performance model.

Cluster analysis is a technique used for identifying groups of the same nature or similarity in larger data sets. Of the many clustering analysis methods, there exists two basic categories: the hierarchical method and the optimization method. The principles underlying the methods are based on measuring the "similarity" or "dissimilarity" of the data objects in the database.

The data objects in this project are the collected pavement sections. The attributes of the pavement sections used for measuring the "dissimilarity" are the changes in pavement age, change in pavement serviceability or performance (ΔPCI), the overall pavement structural depth in terms of the equivalent thickness, H_e, subgrade modulus, M_s and traffic loading history in terms of the accumulated ESALs, N.

The first step in the cluster analysis is to prepare the above data in a matrix form so that each row defines an object, and each column represents a variable. A complete listing of the pavement attributes is given in Appendix C.

The next step is to select a summary statistic for measuring the dissimilarity or the "distance" between the objects. The Euclidean method is the most commonly used for defining the distance. It is calculated by the following equation [Everitt 1993]:

$$d_{ij} = \sqrt{\sum_{k=1}^{p} (x_{ik} - x_{jk})^2} \qquad (4.10)$$

where: i and j represent the objects in the data file. Here they represent any two pavement sections, and k =1,2, ... p is the variable(s) used in the cluster analysis.

The calculation was performed with the SYSTAT™ statistical software package. Among the many methods used in the analysis, the single linkage method, the Ward minimum variance method (both are of the hierarchical type) and the k-means method (optimization method) produced consistent results. A tree diagram generated from the Ward method is given in Appendix D in which the section ID starting with letter "N" represents a pavement section from the North Region or the Northwest region, and the pavement sections starting with other letters are from the Eastern, Central or Southwest regions.

The results of the cluster analysis indicated a largest recognizable group (12 sections) from the Northern and Northwestern Regions in a cluster. No other apparent pattern can be found from the output. It is considered that the cluster analysis result indicated a potential climatic effect on the pavement performance. Because of limitations in the database a climatic variable was not included in the cluster analysis. In future studies, however, the climatic data, such as the freezing index, freeze-thaw cycles,etc., could be included in the analysis.

Although the cluster analysis result is not clear cut (ten other sections from the Northern and Northwest Regions are mixed with South Ontario sections), it is reasonable to conclude, applying engineering judgment, that the data from the Northern and the Northwestern Regions can be pooled together in one group and the data from the Southwest, Central and Eastern Regions be put into another group. The cluster analysis thus provides an approximate grouping method and this result is used in calibrating the OPAC pavement performance model.

From the tree diagram it is found that three more subgroups can be further identified in the Southern Ontario group. As stated in the following section, the strategy of using smaller groups for the model calibration was not successful.

4.1.5 Fitting the Pavement Performance Prediction Model

As stated in Section 4.1 for the overall model updating strategy, the curve fitting is focused on the environmental related part of the OPAC model. Equations (4.1), (4.5) and (4.6) are put together and rearranged as:

$$(1-\frac{1}{1+\beta W_s})(1-e^{\alpha Y}) = 1-\frac{P+P_T}{P_0} \quad (4.11)$$

with the variables defined in Section 4.1.1. The model updating is in effect to calibrate coefficients β and α based on the observed PCI values (for "P" in the equation) and initial performance P_0. The portion of the performance loss due to traffic, P_T, is calculated using Equation (4.4)[6] based on the collected traffic data. Applying the clustering results, the database is divided into two groups. The Southern Ontario group includes pavement sections in the Southwest, Central and Eastern MTO regions, while the Northern Ontario group covers the Northern and Northwestern regions. The same SYSTAT computer package is used for the non-linear regression analysis for the two groups with Gauss-Newton method. The SYSTAT output of the regression analysis results are given in Appendix E. Two sets of new coefficients are acquired, each for one group, as given in Table 4.3.

Table 4.3 Summary Results of Regression Analysis

Parameter	Southern Ontario	Northern Ontario	Existing OPAC Model
β	12.7211	10.5478	2.3622
α	-0.0329	-0.0415	-0.06
R^2	0.707	0.866	N.A.
SSE[7]	2.966	0.383	South 3.262 North 0.905

By inspecting the regression result, a large difference in the magnitude of β is found between the existing OPAC and OPAC 2000. The following aspects in the new database are considered to have contributed to the difference:

1. The Brampton road tests used only one geographical location in Central Ontario and only 36 short test sections. The new coefficients of the Southern Ontario model are based on 77 in-service pavement sections which are spread in all the regions in Southern Ontario,
2. The length of the data series of the Brampton road tests was 8 years, while the database used in developing the OPAC 2000 performance models contains observed performance data of up to 20 years,

[6] A factor of 10 is used to convert ΔRCI_T into P_T (i.e., PCI scale).

[7] SSE - Sum of Squares of Residuals (Errors)

3. For the coefficients of the Northern Ontario model, the site location and the length of data series are considered the main reasons for the difference.

The regression result shows that with the help of cluster analysis the new model for Northern Ontario reduces the prediction error substantially, and the one for Southern Ontario reduces the prediction error to a certain extent (by regression, the SSE value reduced 58% and 9%, respectively). Some sample plots of observed PCI values versus predicted PCI's by both the existing OPAC and OPAC 2000 models are given in Appendix F, in which the first three are sections from Southern Ontario, and the next three are sections from Northern Ontario.

Further tests based on the traffic and structural inputs from the 94 pavement sections in the database indicated that when the Northern Ontario model is used in the Southern Ontario group, the average initial service life is reduced by 3 to 4 years.

Another set of regression analyses was performed on some smaller data groups generated from subdividing the Southern Ontario group and the Northern Ontario group, but the improvement to the prediction was not significant. It was therefore decided to use the coefficient values resulting from the analysis based on the two groups (as presented in Table 4.3) for flexible pavement performance prediction in OPAC 2000.

4.2 Reliability Analysis

One of the shortcomings of the existing OPAC is the lack of a means for quantitatively assessing the reliability of pavement design alternatives. Because of the variability of pavement material properties and the lack of accurate measurement of traffic loads and environmental factors, pavement performance prediction can not be precise. Therefore, decisions on pavement design have to be made under conditions of uncertainty, and it is the duty of the pavement engineer to estimate the level of uncertainty that is associated with his/her designs. and report it to the decision maker. OPAC 2000 provides a tool for this estimation based on standard engineering reliability principles.

4.2.1 Reliability Concept in Pavement Design

The formal definition of reliability as associated with pavement design is given in the AASHTO Guide (Chapter 4, Part I) [AASHTO 1993]. A simplified statement is that reliability is the probability that the pavement will provide a certain level of performance over the design period. The following equation is used for calculating reliability [Kenis 1977]:

$$R = P\,[P_f \geq P_t] \tag{4.12}$$

where: R represents reliability, P_f is the serviceability index (PCI, in OPAC 2000) at a given year, and P_t is the minimum acceptable serviceability level (terminal PCI). This concept is described in Figure 4.1. The shaded area in the figure represents the reliability, or the probability that the performance of the pavement at the given year will be equal to or higher than the minimum acceptable level.

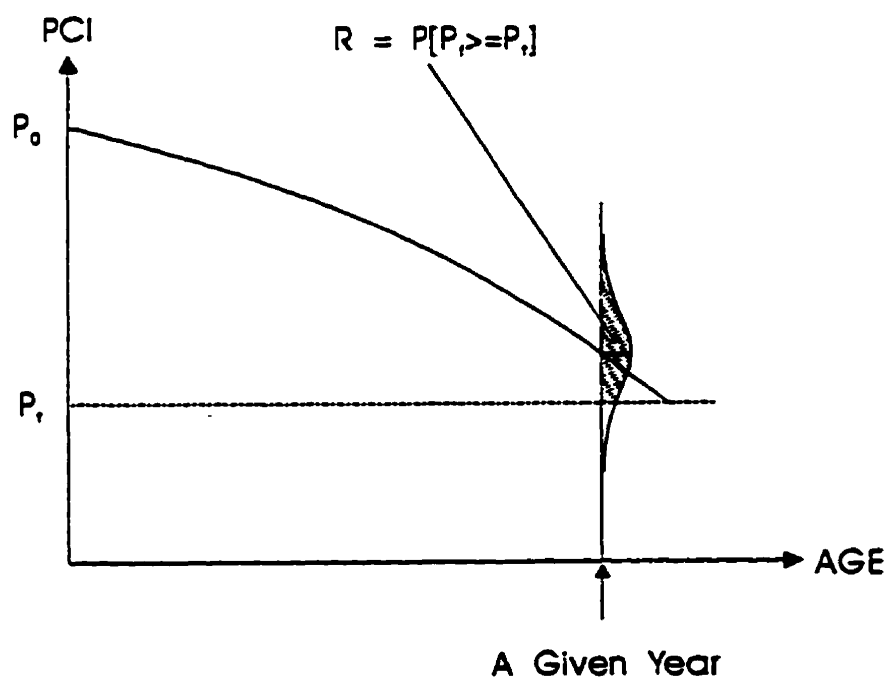

Figure 4. 1 Concept of Pavement Design Reliability

The reliability of the predicted pavement service life can be defined using the same concept, where the reliability is the probability that the pavement being designed will have a service life equal to or longer than the specified minimum service life requirement.

There are two basic sources of uncertainty: (1) the idealization of design inputs, and (2) the error incorporated in the regression model. To account for the uncertainty, the associated variables need to be treated as random variables instead of variables with definite values. In practice the design variables are assumed to be normally distributed about their mean values and variances.

In OPAC 2000 variables considered to contribute to the first type of error include the estimated ESAL applications (N), the GBE's (a_i) of the paving materials, the subgrade modulus (M_s) and the initial performance level (P_0). The model variance (σ^2) from the regression analysis is used to account for the second type of error. The method of predicting pavement performance based on reliability analysis is described in detail in Section 4.3.

4.2.2 Selecting Design Reliability Level

In operating OPAC 2000, a terminal PCI value and a reliability level need to be specified so that design alternatives with reliability lower than the specified level will be rejected. The reliability level selected for pavement design should comply with the Ministry's policy. A set of suggested levels of reliability is given in the AASHTO Guide. They are listed in Table 4.4 for reference.

Table 4.4 Suggested Levels of Reliability by AASHTO (After [AASHTO 1993])

Functional Classification	Urban		Rural	
	Range	Mean	Range	Mean
Interstate and Other Freeways	85 - 99.9	92.5	80 - 99.9	90
Principal Arterials	80 - 99	89.5	75 - 95	85
Collectors	80 -95	87.5	75 - 95	85
Local	50 - 80	65	50 - 80	65

In OPAC 2000, for a specified minimum required initial service life, a higher design reliability level will result in a stronger pavement structure, and hence a higher initial construction cost. A strong initial pavement structure, however, may not necessarily mean a higher total cost because of the potential reduction in the future rehabilitation cost and road user cost. A sensitivity analysis subroutine is included in the system to help assess the impact of different reliability levels.

It should be noted that if the design period (analysis period) includes several cycles of construction/rehabilitation, the compound reliability will be lower than that specified for individual design stages. For example, if the design reliability level is selected as 0.90 and the design period includes one new construction period and two rehabilitation periods, the overall reliability would be 0.90 × 0.90 or 0.81 (assuming the second rehabilitation has not yet reached the end of its service life). In general, the overall reliability can be calculated as:

$$R_{overall} = (R_{individual})^{n-1} \tag{4.13}$$

where: n is the number of constructions/rehabilitations in the design period. This concept is described in the 1993 AASHTO Guide (Part I, Chapter 4) [AASHTO 1993].

4.3 Structural Analysis Procedure

This section describes the flexible pavement design procedure in OPAC 2000. Emphasis is placed on the input data processing with respect to the design criteria, layered material properties, subgrade type and condition, and traffic loading calculation method. The pavement performance analysis procedure and reliability analysis procedure described in this section apply to both the new flexible pavement design and the flexible pavement overlay design.

4.3.1 Information Required for Structural Analysis

Three categories of input data are required by the structural analysis procedure: project data and performance criteria, data for structural analysis and data for economic analysis. Project and performance data include the project ID, the location, length and cross-sectional information, the pavement performance standard, required initial pavement life, reliability, etc. Structural analysis related data include thickness and modulus of pavement materials, subgrade modulus and the expected traffic loading. Inputs related to economic analysis include the funding for construction, discount rate, unit costs, maintenance cost and the cross-sectional data, etc. This section will be focused on the structural related inputs. Economic analysis inputs are subsequently discussed.

4.3.1.1 Pavement Material and Subgrade Data

The thicknesses of pavement layers provide inputs in terms of a set of minimum and maximum layer thickness limits, and an incremental amount of thickness which are used in OPAC 2000 to generate pavement design alternatives. The strength parameters are the GBE values from the MTO Pavement and Rehabilitation Manual [MTO 1990] as previously described.

As in the existing OPAC, a future overlay thickness is also needed for calculating life cycle costs. This thickness is not a design output, but a value estimated by the design engineer. The thickness will be added to the pavement structure if the analysis period (design period) is greater than the initial pavement life, and the performance curve reaches the minimum acceptable level (according to the specified reliability). For clarifying the terminology, this estimated thickness is referred to as "future overlay thickness" in the thesis, as opposed to the designed new overlay thickness; the new overlay design procedure is described in Section 4.3.3.

The subgrade modulus used in flexible pavement designs is given in Table 4.5 (excerpted from [MTO 1990] and converted into SI units). Compared to the existing OPAC, the strength parameters (GBE's and M_r) are not treated as definite quantities, but are used as "mean values" accompanied by the associated errors (in terms of the percentage of the mean) which are estimated by the user as required by the reliability analysis.

Table 4.5 Typical Subgrade Coefficients M_s (MPa, after [MTO 1990])

Subgrade Condition	Granular-Type Materials	Sandy Silt and Clay Till: Silt < 40 v. fine sand & silt < 45	Sandy Silt and Clay Till: Silt 40-50 v. fine sand & silt 45-60	Sandy Silt and Clay Till: Silt >50 v. fine sand & silt > 60	Lacus-trine Clays	Varved & Leda Clays
Good	79.3	48.3	41.4	31.0	37.9	31.0
Fair	72.4	41.4	34.5	27.6	34.5	24.1
Poor	62.1	37.9	31.0	24.1	27.6	17.2

4.3.1.2 Traffic Data

The OPAC 2000 model requires estimating the number of traffic loads to be carried by the pavement in terms of the standard 80 kN equivalent single axle load, i.e., ESALs. The accumulated ESAL number is used in Equation (4.4) as the N value to estimate the traffic associated pavement performance loss. The new ESAL calculation method is based on the following equations [Hajek 1995b]:

For geometric (exponential) growth,

$$N = \sum_{j=1}^{Y} \sum_{i=1}^{n} [(AADT \cdot T \cdot t_i \cdot TF_i \cdot DF \cdot LDF \cdot DAYS)(1+GR)^{j-1}] \tag{4.14}$$

For linear growth,

$$N = \sum_{j=1}^{Y} \sum_{i=1}^{n} \{(AADT \cdot T \cdot t_i \cdot TF_i \cdot DF \cdot LDF \cdot DAYS)[1+GR(j-1)]\} \tag{4.15}$$

where:

N	=	total number of ESALs accumulated in the design lane after the latest construction (or overlay),
Y	=	number of years since the recent construction,
AADT	=	initial year average annual daily traffic
n	=	number of truck classes
T	=	truck fraction in the total AADT
t_i	=	proportion of the truck population which belongs to truck class i
TF_i	=	Truck Factor for truck class i. Here either the FHWA 13 class vehicle classification schemes (omitting the first 3 classes) or the simplified four-class scheme [Hajek 1995b] can be used. Truck Factors for both schemes are given in Tables 4.6 and 4.7,
DAYS	=	days per year for truck traffic. A default value of 300 is used for Ontario,
LDF	=	lane distribution factor, used to account for the truck traffic in the design lane. Values of LDF can be found in Table 4.8,

GR = traffic growth rate, can be either geometric or linear.

Table 4.6 Typical Truck Factors for Simplified Vehicle Classification [Hajek 1995b]

Major Truck Classes	Truck Factor, TF	Range of Truck Factor
2 and 3-axle trucks	0.40	0.05-0.90
4-axle trucks	2.00	0.2-0.4
5-axle trucks	1.20	0.3-3.5
6 and more axle trucks	5.10	2.0-6.5

Table 4.7 FHWA Vehicle Classes and Typical Truck Factors [Hajek 1995b]

Vehicle Classes	Truck Factor, TF	Typical TF
1	Motorcycles	0
2	Passenger cars including cars pulling trailers	0
3	Other two-axle four-tire single unit vehicles	0
4	Buses with two or more axles	1.10
5	Two-axle six-tire single unit trucks	0.30
6	Three-axle single unit trucks	0.80
7	Four or more axle single unit trucks	4.00
8	Four or less axle single trailer trucks	0.50
9	Five-axle single trailer trucks	1.20
10	Six or more axle single trailer trucks	3.50
11	Five or less axle multi-trailer trucks	1.50
12	Six-axle multi-trailer trucks	5.10
13	Seven or more axle multi-trailer trucks	4.10

The accumulated ESAL's (N) thus calculated is used as the mean value in the input of OPAC 2000 pavement design system. The user is asked to estimate and input the possible error of N which is used in the reliability analysis.

Table 4.8 Lane Distribution Factor (LDF) [Hajek 1995b]

Number of lanes in one direction	AADT	LDF
1	all	1.00
2	< 15,000	0.90
	≥ 15,000	0.80
3	< 15,000	0.85
	25,000 - < 40,000	0.80
	≥ 40,000	0.70
4	< 40,000	0.80
	≥ 40,000	0.70

4.3.2 New Flexible Pavement Design

The structural analysis of flexible pavements starts with generating design alternatives (various layer combinations) based on the specified thickness limits and the required increments. The results are organized in an n-dimensional array, where: n is the number of layers. For new overlay designs this n equals to 1. For each pavement design alternative, the analysis procedure includes the following steps:

1. Calculate mean PCI values based on the yearly accumulated ESALs
2. Calculate variance in PCI due to errors in design variables (σ_v^2) based on the variance of P_0, GBE's, M_s and N,
3. Calculate yearly pavement performance for given reliability,
4. Determine the pavement life period, and
5. Future overlay analysis.

These steps of the structural analysis are repeated for the future overlay period until the pre-specified analysis period is reached.

4.3.2.1 Calculating Mean PCI Values Based on the Yearly Accumulated ESALs

To calculate pavement performance, PCI, one of the models developed in Section 4.1 is selected according to the location of the project (Southern or Northern Ontario). The result of this calculation will be used later in the process of calculating PCI values for the given reliability. The procedure is described as follows:

Step 1: Calculate the equivalent granular thickness of the designed pavement structure:

$$H_e = \sum h_i GBE_i m_i \tag{4.16}$$

where:

h_i	=	thickness of layers of a design alternative,
GBE_i	=	Granular Base Equivalency factor of the layers,
m_i	=	drainage coefficient of the material[8].

Step 2: Calculate the Odemark subgrade deflection of the designed pavement structure:

$$W_s = 1000 \times \frac{p}{2M_s Z\sqrt{1+\left(\frac{a}{Z}\right)^2}} \tag{4.17}$$

where:

p	=	standard wheel load (i.e., 40 kN on a dual tire)
M_s	=	modulus of the subgrade (MPa)
Z	=	$0.9H_e\sqrt[3]{\frac{M_2}{M_s}}$
M_2	=	modulus of the equivalent granular base material (average 345 MPa)

[8] Drainage coefficients are set to a default value of 1 at the current stage. MTO requires provision of the ability to use the drainage coefficients in the future when specific study results are made available.

a = radius of loaded area (i.e., approximately 163 mm for an equivalent circular imprint of a dual tire).

Step 3: Calculate the performance loss due to traffic:

$$P_T = 10\times(2.4455\Psi + 8.805\Psi^3) \quad (4.18)$$

where:

Ψ = $3.7239 \times 10^{-6} \times W_S^{6} \times N$ (for W_S calculated in mm)

N = number of (80 kN) ESAL applications,

Step 4: Calculate the performance loss due to environment:

$$P_E = P_0\left(1 - \frac{1}{1+\beta W_S}\right)\left(1 - e^{\alpha Y}\right) \quad (4.19)$$

where:

P_0 = initial performance index

W_S = as previously defined.

Y = number of years in service.

α and β in the model are determined according to Table 4.3.

Step 5: Calculate the pavement performance index:

$$P = P_0 - P_T - P_E \quad (4.20)$$

where: P_0 is the initial pavement performance index, and P_T and P_E are the performance losses due to traffic and environment, respectively.

4.3.2.2 Calculating σ_v^2 Based on the Variance of P_0, GBE's, M_s and N

Equation (4.12) requires calculations of the mean and variance of the dependent variable based on the distributions of independent variables. For nonlinear models, such as the one in OPAC 2000, it is often difficult to solve directly because of the integration involved in calculating probabilities. The second moment approximation method [Ang 1984] is used for calculating the variance of the pavement performance index PCI due to the error in the input (σ_V^2) based on variances in GBE, M_s, N and P_0:

$$\sigma_V^2 = \sum \left(\frac{\partial P}{\partial X_i} \right)^2 \sigma_{X_i}^2 \qquad (4.21)$$

where:

σ_V^2 is the variance in PCI due to errors in design variables. X is the vector of design variables P_0, H_e, M_s and N. σ_{Xi}^2 is the variance of the design variables. $\partial P/\partial X_i$ is the partial derivative of PCI with respect to one of the design variables. The partial derivatives of pavement performance (P) with respect to each of the individual design variables P_0, H_e, M_s and N are given in Appendix G.

and

$$\sigma_{Xi} = COV_{Xi} \times X_i \qquad (4.22)$$

where both COV_{Xi} (coefficient of variation of the i^{th} variable) and X_i (mean value of the i^{th} variable) are from the inputs.

4.3.2.3 Calculating Yearly Pavement Performance for Given Reliability

The pavement performance Index (PCI) is calculated on a yearly basis using the equations in Section 4.1. The yearly pavement performance index PCI_f with a given reliability level is determined as:

$$PCI_f = PCI + z_R \sigma_{PCI} \qquad (4.23)$$

where:

$$\sigma_{PCI} = \sqrt{\sigma_V^2 + \sigma_M^2} \text{ [Alsherri 1988]} \qquad (4.24)$$

σ_M is the standard deviation corresponding to the prediction errors due to regression, which equals 7.027 and 4.661 for Southern Ontario (Southwest, Central and Eastern Regions) and Northern Ontario (Northern and Northwestern Regions), respectively.

and

z_R is the standard normal deviate corresponding to the design reliability level (R). To facilitate programming, the "Inverse Normal Probability Integral" method is used in determining z_R [Abramowitz 1964]:

$$z_R = [\mathrm{sgn}(v)] \times \left(t - \frac{c_0 + c_1 t + c_2 t^2}{1 + d_1 t + d_2 t^2 + d_3 t^3} \right) \qquad (4.25)$$

where:

$v = 0.5 - R$

$t = \sqrt{-2\ln(0.5 - |v|)}$

$c_0 = 2.515517$, $c_1 = 0.802853$, $c_2 = 0.010328$, and

$d_1 = 1.432788$, $d_2 = 0.189269$, $d_3 = 0.001308$.

The following table contains typical z_R values for different reliability level (R):

Table 4.9 Typical z_R values for different reliability levels

R	z_R
95%	- 1.645
90%	- 1.282
85%	- 1.036
80%	- 0.842
75%	- 0.674

Suggested design reliability levels and some comments on the effect of using different design reliability levels with OPAC 2000 are given in Section 4.2.2.

4.3.2.4 Determining Pavement Life Period

The calculated PCI_f is compared with the minimum acceptable level PCI (P_t). The life of the pavement is determined as the time required for PCI_f (for given reliability) to reach P_t. The pavement life period before the first overlay is the initial life. Design alternatives with an initial pavement life shorter than the specified value are discarded. For other design alternatives the program continues to perform future overlay analysis.

It is possible that there is no feasible design alternative available due to improper input of either the range of layer thickness or the budget limit. The following method is used to deal with the problem. After generating the design alternatives, the program[9] starts analyzing the design alternative with the maximum structural thickness and determines its initial life. This initial life is then compared with the required initial life from the input, as follows:

1. If the calculated initial life is shorter than the required, the program will stop and give a message telling the user that there is no feasible design alternative for the current set of inputs and remind the user to adjust the input and try again,

2. Otherwise the program goes on to analyze the design alternative with the minimum structural thickness and determines its initial agency cost. This initial cost is then compared with the available funding level from the input. If the initial cost is higher than the available funding level, the program will stop and give the same message as in "1",

The situation in "1" indicates that the layer thickness specified by the user is too low or the selected subgrade is too weak for the traffic condition in the design. The situation in "2" indicates that the budget limit specified for the design is too low. The program will save a file as a table which shows all the inadequate design alternatives with the reason why they are rejected.

3. If the situations in either "1" or "2" do not appear, the normal analysis process will begin from "Calculating Mean PCI Values".

4.3.2.5 Future Overlay Analysis

For performing the life-cycle cost analysis, the specified future overlay thickness is added on the pavement structure at the end of each analysis cycle. The above calculations are repeated with the following modifications:

Equation (4.16) for calculating H_e is replaced by:

[9] The "program" refers to the OPAC 2000 computer package as subsequently described, which incorporates the method described herein.

$$H_e = h_o GBE_o + \sum k_i h_i GBE_i m_i \tag{4.26}$$

where:

h_o	=	future overlay thickness specified in the input,
GBE_o	=	GBE of the future overlay material,
h_i	=	layer thicknesses of the design alternative,
GBE_i	=	GBE of the layers in the design alternative (refer to the MTO Pavement Design and Rehabilitation Manual [MTO 1990]),
m_i	=	drainage coefficients[10]
k_i	=	overlay equivalency reduction factors for asphalt surfacing, base and subbase, respectively. They are determined by following equations[11][Jung 1975]:

For asphalt layers:

$$k_i = 0.44 + 0.0068\ PCI_f \tag{4.27}$$

For granular layers:

$$k_i = 0.8 + 0.3125\ (0.38 - W_s) \tag{4.28}$$

PCI_f in Equation (4.27) is the pavement performance index before the overlay. Here it may be slightly higher than the minimum acceptable performance level due the fact that PCI_f is calculated on a yearly basis.

For calculating the accumulated traffic load in future overlay analysis, Equation 4.14 (or 4.15, if the linear growth is chosen) is modified in the way that the ESALs occurred before the future overlays are excluded.

More overlays are triggered when the predicted PCI_f reaches P_t. The process continues until the total number of years reaches the analysis period (AP). The structural analysis of one design alternative is finished at this point. The program will go back to "Design Alternative Generator", and the calculations are repeated for another design alternative.

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[10] Drainage coefficients are set to a default value of 1 at the current stage. MTO requires provision of the ability to use the drainage coefficients in the future when specific study results are made available.

[11] The equations are based on the source report as given in the reference with some modifications.

After all the design alternatives have been analyzed, the analysis outputs including structural depth, performance history and pavement life are then used as inputs to the economic analysis module for life-cycle cost analysis.

4.3.3 Overlay Design on Flexible Pavement

The same performance models and procedure as those of new flexible pavement designs are used in the new overlay designs for flexible pavements. The only change is with the calculation of the equivalent structure thickness H_e. Equation (4.16) is changed to:

$$H_e = h_0 GBE_0 + \sum h_{exi} GBE_{exi} m_{exi} \tag{4.29}$$

where:

h_0	=	new overlay thickness,
GBE_0	=	GBE of the new overlay material,
h_{exi}	=	layer thicknesses of the layers in the existing pavement structure,
GBE_{exi}	=	GBE of the layers in the existing pavement structure (refer to the MTO Pavement Design and Rehabilitation Manual [MTO 1990] for the coefficient of the existing pavement materials),
m_{exi}	=	drainage coefficients

The user is asked to identify the existing pavement layers as well as the new overlay layer and to input the GBE's for both new and old materials with the associated estimated errors. The process of structural alternative generation is modified so that design alternatives are generated by varying only the thickness of the new overlay layer. In the event that there is no feasible overlay design alternative available for the specified input, the same message should be given as mentioned earlier in the section of "Determining Pavement Life", and the user is prompted to modify the input.

For performing the life-cycle cost analysis, design alternatives satisfying the requirements of both the initial design life and the funding level will be further analyzed for future overlays using the same procedure as in new pavement designs.

4.4 Sample Analysis

A six-lane highway is taken as an example of the structural analysis using the OPAC 2000 flexible pavement analysis procedure. The structural-related inputs are given in Table 4.10:

Table 4.10 Sample Project Data of Flexible Pavement Analysis

Layers and Site Information		Other Design Criteria	
Hot Mix Asphalt Concrete	50 (mm)	Lane width	3.75 (m)
Hot Mix Asphalt Concrete	110 (mm)	Div'd/Und	Undivided
Granular Base	150 (mm)	Initial performance index (P_0)	95
Granular Subbase	375 (mm)	Performance index after future overlays (P_{future})	90
Future Overlay	75 (mm)	Minimum acceptable performance index (P_t)	50
Mill-off depth before future overlay	10 (mm)	Reliability (R)	0.9
Subgrade Strength (M_s)	34.5 (MPa)	COV*	0.1

* Coefficient of variation of design variables GBE, M_s and N

The traffic load anticipated on the above pavement structure is 12,500 initial AADT increasing at a rate of 4% per year. There is 17% of total trucks in the traffic flow, 40% of it is two and three axle trucks, 30% four axle trucks, 20% five axle trucks and 10% six and more axle trucks. The analysis period is 30 years. The foregoing traffic inputs translate into a yearly 80 kN equivalent single axle load (ESAL) of 409,116 in the first year and 1,200,837 by year 30.

Applying the OPAC 2000 structural analysis procedure (assuming this highway is in Southern Ontario), the equivalent granular thickness (H_e) of the above pavement structure from Equation (4.16) is 639 mm, and the Odemark subgrade deflection (W_s) from Equation (4.17) is 0.464 mm. The yearly pavement performance is predicted as shown in Table 4.11.

The result in Table 4.11 shows that the pavement structure will have an initial service life of 12 years with a 90% reliability. It requires future overlays at Year 13 and Year 22. By the end of the 30 year analysis period the performance index will be about 60 PCI.

Table 4.11 Sample Flexible Structural Analysis Results

1st Period		2nd Period		3rd Period	
Year	PCI	Year	PCI	Year	PCI
0	85.7	13	80.7	22	80.7
1	82.7	14	77.6	23	78.0
2	79.8	15	74.4	24	75.2
3	76.9	16	71.3	25	72.6
4	74.1	17	68.0	26	69.9
5	71.3	18	64.7	27	67.3
6	68.5	19	61.2	28	64.7
7	65.7	20	57.4	29	62.1
8	62.8	21	53.3	30	59.5
9	60.0				
10	57.0				
11	54.0				
12	50.8				

CHAPTER 5 RIGID PAVEMENT DESIGN MODULE

Before starting development of the rigid design module for the OPAC 2000 package, an investigation was made on the available rigid design methods/packages (see Chapter 3). As indicated previously the AASHTO rigid pavement design method was selected as the basis of the OPAC 2000 rigid pavement design module[12]. The following rigid (PCC) pavement design types are included in the rigid pavement design module: new rigid or PCC pavements, bonded PCC overlay on existing PCC pavements, unbonded PCC overlay on existing PCC pavements, asphalt concrete (AC) overlay on rigid pavements (PCC) and AC overlay on AC-overlaid PCC pavements (AC/PCC),

To help determining the feature that should be included in the rigid design module of OPAC 2000, a detailed investigation of DARWin™ 2.0 and PAS 5.0 was made. Although there is no significant difference between DARWin and PAS, there are some variations in the user interface and calculation functions. DARWin is operated in a Microsoft Windows™ environment. The Windows environment provides an integrated system performing multiple tasks and supports more active screens. PAS, however, is designed for use under the conventional DOS environment which supports only one active screen. Another difference is with their power of calculations. Generally, DARWin has more calculation functions than PAS does.

The limitations of both packages are very similar. First, both of them do not have road user cost elements in their life cycle cost streams. This is due to the lack of the capability to predict pavement performance change, which is an essential requirement for estimating the road user cost (vehicle operating cost and the user delay cost). Another limitation, or inconvenience to the Ontario users, is that both packages are developed in Imperial units. In other words, SI units, the official units used in Canada, are not incorporated. As a result, it would be desirable for Ontario pavement engineers to have a rigid pavement design package which accepts SI units.

With the preceding considerations, the rigid design module in OPAC 2000 was developed with the following features. The first feature of the module is the capability to predict rigid pavement performance change over time, and the result is used in determining the pavement life and road user costs. The second feature is that it offers a user interface in SI units. The most unique feature, however, is that the rigid

[12] In the absence of sufficient Ontario rigid pavement performance data, the AASHTO rigid pavement design equation is used without modification. Users of OPAC 2000 are encouraged to check their designs with local practice.

pavement design module is organised in line with the OPAC pavement design philosophy; that is, firstly generating design alternatives, then carrying out structural analysis in terms of performance predictions, followed by economic analysis which gives the life-cycle costs, and finally ranking the analysis results with certain criteria in the output.

This chapter describes the modifications made to the AASHTO rigid pavement design method and the organization of the OPAC 2000 rigid pavement design modules. As a result of the modifications the newly developed rigid pavement design module incorporates both the structural and economic analyses, offers greater flexibility to the pavement design engineer and is able to give design reports on a number of design alternatives, instead of working only on one design alternative as is the case with the DARWin and PAS systems.

5.1 The AASHTO Rigid Pavement Design Equation

Equation (5.1) from Part I of the 1993 AASHTO Guide [AASHTO 1993] is the basic formula for rigid pavement structural analysis in OPAC 2000. Since all the design inputs required by the equation are in Imperial units, the input parameters in OPAC 2000 are converted from the SI units into Imperial units at the beginning for the analysis. After the structural analysis the results in Imperial units are converted back into SI units in the design outputs.

$$\log_{10} W_{18} = Z_R \times S_0 + 7.35 \times \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{P_0 - P_f}{4.5 - 1.5}\right]}{1 + \frac{1.624 \times 10^7}{(D+1)^{8.46}}}$$
$$+ (4.22 - 0.32 \times P_f) \times \log_{10}\left[\frac{S_c' \times C_d \times \left(D^{0.75} - 1.132\right)}{215.63 \times J\left[D^{0.75} - \frac{18.42}{(E_c / k)^{0.25}}\right]}\right] \qquad (5.1)$$

where:

W_{18}	=	predicted number of 80 kN (18-Kip) equivalent single axle load applications,
Z_R	=	standard normal deviate, see Section 5.3.2.

S_o	=	combined standard error of the traffic estimation and performance prediction,
D	=	thickness of pavement slab, mm (in)
P_0	=	the initial design serviceability index (a factor of 0.05 is used to convert the PCI input into the PSI[13] unit , ranging from 0 to 5),
P_f	=	the serviceability index at a given year (a factor of 0.05 is used to convert the PCI input into PSI, ranging from 0 to 5),
S'_c	=	modulus of rupture for portland cement concrete used on a specific project, MPa (psi),
C_d	=	drainage coefficient,
E_c	=	modulus of elasticity for portland cement concrete, MPa (psi),
k	=	modulus of subgrade reaction, MPa/mm (psi/in), and
J	=	Load transfer coefficient.

Equation (5.1) is used for determining the amount of traffic loading in terms of the total cumulative number of ESALs for a given PCC slab thickness (D), the allowable pavement performance loss (P_0-P_f) and other inputs. In OPAC 2000 the equation is transformed so that for each design alternative the yearly pavement performance index (P_f) is solved for based on the projected yearly traffic. This process is subsequently discussed further.

5.2 Structural Analysis Procedure

There are five rigid pavement design submodules in OPAC 2000 for designing new rigid pavements, bonded and unbonded PCC overlays, AC overlay on PCC pavements and AC overlay on AC/PCC pavements. While the overall structural analysis procedure is similar to that in the flexible pavement design module, the differences will be emphasized.

OPAC 2000 does not include details for joint and reinforcement designs in the rigid pavement design module, but their costs can be included in the economic analysis. For the details of joint and reinforcement design procedures for the three kinds of rigid pavements (jointed plain concrete pavement,

[13] PSI: Present Serviceability Index developed at the AASHO road test, with 5 representing the perfect pavement condition and 0 representing total failure.

JPCP; jointed reinforced concrete pavement, JRCP and continuously reinforced concrete pavement, CRCP) references should be made to the AASHTO Guide [AASHTO 1993].

5.2.1 Data Requirements

Of the three categories of input data, the project ID and performance criteria, most of the structural analysis data and the economic analysis data are organized in the same way as in the two flexible pavement design submodules. Some particular inputs required by the AASHTO rigid pavement design equation are subsequently explained along with corresponding design submodules.

5.2.2 New Rigid Pavement Design

The structural analysis of rigid pavements starts with generating design alternatives (various PCC slab and the subbase layer combinations) within the thickness limits and increments specified by the designer. The results are organized in a 2-dimensional array. For new overlay designs the dimension reduces to 1. For each pavement design alternative, the analysis procedure the includes the following parts:

1. Calculate the yearly accumulated ESALs,
2. Calculate design subgrade reaction k-value,
3. Calculate yearly pavement performance index P_f and determine pavement life, and
4. Carry on future overlay analysis.

As in the flexible pavement design module, a future overlay thickness is needed for the life cycle cost analysis. This thickness is not a design output, but a value estimated by the user. The thickness will be added to the pavement structure if the analysis period (design period) is greater than the initial pavement life, and the performance curve reaches the minimum acceptable level (according to the specified reliability). In OPAC 2000 the future overlay material can be either asphalt concrete or portland cement concrete for the design types of "new rigid pavement", "bonded PCC overlay" and "unbonded PCC overlay"; while it can only be asphalt concrete for "AC overlay on PCC pavement" and for "AC overlay on AC/PCC pavement" designs.

5.2.2.1 Calculate the Yearly Accumulated ESALs

Based on the input AADT value, the amount of truck traffic and the distribution of the truck classes, the same method as described for the flexible pavement design (see Chapter 4) is used for calculating accumulated ESALs in rigid pavement designs. The accumulated ESAL (N) in Equations (4.14) and (4.15) becomes W_{18} in Equation (5.1).

5.2.2.2 Calculate Subgrade Reaction k-Value for Each Design Alternative

The subgrade reaction k is a function of the strength of the road bed soil, the depth to rigid foundation (bedrock) the thickness of concrete slab and the thickness of the subbase. The term subbase is used by AASHTO, which is also referred to as "base" in other design methods. The calculation is accomplished by using the procedure stated in the AASHTO Guide. Basically, the k-value is determined under two different conditions: without the effect of bedrock (depth to bedrock exceeds 3m (10 ft)), and with the effect of bedrock (depth to bedrock is less than 3m (10 ft)).

(1) Without the effect of bedrock

In the case where a subbase is used, the composite modulus of subgrade reaction without bedrock effect is defined as (1986 AASHTO Guide, Volume II) [14]:

$$\begin{aligned}\log(k_{inf}) =& -2.807 + 0.1253\,[\log(D_{sb})]^2 + 1.062\log(M_R) \\ &+ 0.1282\log(D_{sb}) \times \log(E_{sb}) - 0.4114\log(D_{sb}) \\ &-0.0581\log(E_{sb}) - 0.1317\log(D_{sb}) \times \log(M_R)\end{aligned} \qquad (5.2)$$

where:

k_{inf} = subgrade reaction without rigid foundation, MPa/mm (psi/in)

D_{sb} = subbase thickness, mm (in)

M_R = average roadbed soil modulus after considering the seasonal effect, MPa (psi)

E_{sb} = average subbase modulus after considering the seasonal effect, MPa (psi)

[14] As in the 1986 AASHTO Guide, the function "log" denotes the natural logarithm "Ln".

In the case where the slab is directly placed on the subgrade, i.e., without the subbase layer, the composite modulus of subgrade reaction is defined as:

$$k_{MR} = M_R/19.4 \tag{5.3}$$

(2) With the effect of bedrock

When the depth from the top of subgrade to the bedrock is less than 3m (10 ft), the subgrade reaction k with the bedrock effect is defined as:

$$\log(k_{rf}) = 5.303 + 0.0710 \log(D_{sg}) \times \log(M_R) + 1.366 \log(k_{inf,MR}) - 0.9187 \log(D_{sg}) - 0.6837 \log(M_R) \tag{5.4}$$

where:

k_{rf} = subgrade reaction with rigid foundation, MPa/mm (psi/in)

D_{sg} = subgrade thickness, mm (in)

M_R = average roadbed soil modulus after considering the seasonal effect, MPa (psi)

$k_{inf,MR}$ = determined by Equation (5.2) or Equation (5.3)

k_{inf}, k_{MR} and k_{rf} from the above calculations are also called effective k (k_{eff}). To obtain the final design subgrade reaction k-value, k_{eff} is modified by the effect of "loss of support" (LS) of the subgrade:

1. when LS = 0 (i.e., stable subgrade):

$$k = k_{eff} \tag{5.5}$$

2. when LS = 1:

$$k = 0.257\, k_{eff} + 13.991 \tag{5.6}$$

where: k_{eff} = 5 ... 2000.

3. when LS = 2:

$$k = 0.07\, k_{eff} + 7.318 \tag{5.7}$$

where: k_{eff} = 10 ... 2000.

4. when LS = 3:

$$k = 0.017\, k_{eff} + 5.963 \tag{5.8}$$

where: $k_{eff} = 10 \ldots 2000$.

The loss of support, LS, is a unitless parameter which depends on the condition of the material underneath the slab. Table 5.1 shows the schedule of typical LS values [AASHTO 1993].

Table 5.1 Typical Loss of Support Values (LS)

Type of Material	Modulus (MPa)	Loss of Support
Cement treated granular base	7000-14000	0-1.0
Cement aggregate mixture	3450-7000	0-1.0
Asphalt treated base	2400-7000	0-1.0
Bituminous stabilized mixtures	280-2100	0-1.0
Lime stabilized materials	140-480	1.0-3.0
Unbound granular materials	100-310	1.0-3.0
Fine grained or natural subgrade materials	20-280	2.0-3.0

5.2.2.3 Calculate Yearly Pavement Performance Index P_f and Determine Pavement Life Periods

For a particular design alternative, PCC slab thickness D is known. With the calculated subgrade reaction k-value, the yearly accumulated ESALs and other design parameters such as material properties, drainage and reliability given in the input, P_f is the only unknown in Equation (5.1). The yearly P_f value is obtained through a "solve for" routine programmed in the software package.

The result of this calculation provides a series of points which form a predicted performance curve for the pavement design alternative. Using the same procedure as in the flexible pavement design module, the initial pavement life is determined when P_f reaches the minimum acceptable performance level P_t. The feasible design alternatives are determined based on the required initial life and the available funding level. These alternatives are then analyzed for future overlays.

5.2.2.4 Future Overlay Analysis

For performing the life-cycle cost analysis, the specified future overlay thickness is added on the pavement structure at the end of each analysis cycle. The AASHTO "Remaining Life" method is used in determining the effective slab thickness at the time of future overlays [AASHTO 1993]. The analysis is carried out in the following steps:

Step 1: Determining the accumulated ESAL's for $P_f = 1.5$ PSI

According to the AASHTO "Remaining Life" method, this is achieved by plugging $P_f = 1.5$ into Equation (5.1) with the reliability level set to be 50%, then $N_{1.5} = 10^{\log W18}$.

Step 2: Determining the effective slab thickness of the design alternative before the future overlay

$$D_{feff} = (1 - N_t / N_{1.5})^{0.165} D \qquad (5.9)$$

where:

N_t	=	the accumulated ESAL's corresponding to $P_f = P_t$,
D	=	the PCC slab thickness of the design alternative.

when $N_t \geq N_{1.5}$,

$$D_{feff} = 0.5\ D \qquad (5.10)$$

Step 3: Determine the total required slab thickness after future overlays

$$D_T = D_{fo} + D_{feff} \qquad (5.11)$$

where:

D_{fo}	=	thickness of future PCC overlay from the input.

When asphalt concrete (AC) is chosen as the future overlay material, the AC thickness (D_{AC}) from the input needs to be converted to the equivalent PCC thickness (D_{fo}) using the following equation:

$$D_{AC} = 2.2233\ D_{fo} - 0.1534\ D_{fo}^2 + 0.0099\ D_{fo}^3 \qquad (5.12)$$

The equivalent PCC thickness (D_{fo}) needs to be solved for using Equation (5.12). Once D_{fo} is obtained, Equation (5.11) is used to calculate the total slab thickness.

Step 4: Analyze pavement performance and pavement life

The new slab thickness obtained from Step 3 is used in Equation (5.1) to determine the pavement performance and pavement life after future overlays. As in the flexible pavement design module, the calculation of the accumulated ESALs needs to be modified to exclude the ESALs which occurred before future overlay(s).

More overlays are triggered when the predicted P_f reaches P_t. The process continues until the total number of years reaches the analysis period (AP). The structural analysis of one design alternative is finished at this point. The program will go back to "Design Alternative Generator", and the calculations are repeated for each design alternative.

After all the design alternatives have been analyzed, the structural depth, performance history and the pavement life period of each feasible design alternative are entered into the economic analysis module for life-cycle cost analysis.

5.2.3 Overlay Designs on Rigid Pavements

OPAC 2000 can be used for four types of new overlay designs: bonded PCC overlay on PCC pavement, unbonded PCC overlay on PCC pavement, AC overlay on PCC pavement and AC overlay on AC/PCC pavement. All the design procedures are based on Part III of the AASHTO Guide [AASHTO 1993] with modifications so that the design analysis follows the OPAC 2000 procedure. This section gives the detailed calculation procedure of the four types of overlay thickness designs. It should be emphasized that the importance of a pavement condition survey, the guidelines on determining the feasibility of the overlay strategy, pre-overlay repairs and reflection crack control, etc., as documented in the AASHTO Guide, should not be overlooked.

5.2.3.1 Bonded PCC Overlay on PCC Pavement

Bonded PCC overlay requires a reliable bond between the overlay layer and the existing PCC surface. The structural analysis is performed in the following steps:

Step 1. Determine the required total slab thickness (D_f) for the future traffic

The total PCC slab thickness D_f for the future traffic and the effective thickness D_{eff} of the existing PCC slab must be determined in order to determine the required overlay thickness. For a given or specified minimum acceptable performance level P_t, D_f can be determined by solving for "D" in Equation (5.1) with a trial and error procedure. The future traffic is the accumulated ESALs (W_{18}) projected for the required initial pavement life.

Some design inputs are different from the ones for new rigid pavement designs. The parameters required by Equation (5.1), such as the modulus of subgrade reaction k, concrete moduli E_c and S'_c and slab load transfer J, should represent the property of the existing pavement rather than that of the new rigid pavement. These inputs can be entered directly or be computed by the backcalculation procedure in OPAC 2000 if the deflection data is available. The backcalculation procedures are described in the following sections.

Step 2. Determine the effective thickness D_{eff} of the existing PCC slab

The effective thickness D_{eff} of the existing PCC slab is determined by the Condition Survey Method:

$$D_{eff} = F_{jc} \times F_{dur} \times F_{fat} \times D \tag{5.13}$$

where:

D_{eff}	=	effective thickness of the existing slab, mm (in)
F_{jc}	=	joint and cracks adjustment factor,
F_{dur}	=	durability adjustment factor, and
F_{fat}	=	fatigue factor
D	=	thickness of the existing slab, mm (in)

It should be noted that the remaining life method is not used here, it is only used for determining the effective thickness D_{feff} in future overlay analysis.

Step 3. Determine the required overlay thickness (D_{ol})

The boned PCC overlay thickness D_{ol} is calculated with the following equation:

$$D_{ol} = D_f - D_{eff} \tag{5.14}$$

where:

D_{ol}	=	PCC overlay thickness, mm (in)
D_f	=	total slab thickness to carry future traffic from Equation (5.1), mm (in)

D_{eff} = effective thickness of existing slab from Equation (5.13), mm (in)

Step 4. Pavement performance, pavement life and future overlay analysis

The procedure for performance, pavement life and future overlay analyses are the same as described in the Section 5.2.2 on New Rigid Pavement Design. The new PCC slab thickness D_f is used in Equation (5.1) for solving the yearly pavement performance P_f.

It should be noted that in new overlay designs the overlay PCC thickness thus acquired will only be the design alternative with the minimum required thickness. The program may generate more design alternatives according to the input overlay thickness boundaries and the increment in order to make comparisons of the life cycle cost based on both agency costs and road user costs.

5.2.3.2 Unbonded PCC Overlay on PCC Pavement

The structural analysis for unbonded PCC overlay design shares the same procedure as in bonded PCC overlay designs. The differences are in Steps 2 and 3 where a different equations are used for determining D_{eff} and D_{ol}:

Step 2. Determine the effective thickness D_{eff} of the existing PCC slab

The effective thickness D_{eff} of the existing PCC slab is determined by the Condition Survey method:

$$D_{eff} = F_{jcu} \times D \tag{5.15}$$

where:

D_{eff} = effective thickness of the existing slab, mm (in)

F_{jcu} = joint and cracks adjustment factor for unbonded PCC overlay,

D = thickness of the existing slab, mm (in)

It should be noted that the remaining life method is not used here, it is only used for determining the effective thickness D_{feff} in future overlay analysis.

Step 3. Determine the required overlay thickness (D_{ol})

The unbonded PCC overlay thickness D_{ol} is calculated with the following equation:

$$D_{ol} = \sqrt{D_f^2 - D_{eff}^2} \tag{5.16}$$

where:

D_{ol} = PCC overlay thickness, mm (in)

D_f = total slab thickness to carry future traffic from Equation (5.1), mm (in)

D_{eff} = effective thickness of existing slab from Equation (5.15), mm (in)

5.2.3.3 AC Overlay on PCC Pavement

The structural analysis for AC overlay design shares the same procedure as in bonded PCC overlay designs. The only change is in Step 3 where a different equation is used for determining the AC overlay thickness D_{ol}:

Step 3. Determine the required overlay thickness (D_{ol})

The AC overlay thickness D_{ol} is calculated with the following equation:

$$D_{ol} = [2.2233 + 0.0099\,(D_f - D_{eff})^2 - 0.1534(D_f - D_{eff})](D_f - D_{eff}) \tag{5.17}$$

where:

D_{ol} = AC overlay thickness, mm (in)

D_f = total slab thickness to carry future traffic from Equation (5.1), mm (in)

D_{eff} = effective thickness of existing slab from Equation (5.13), mm (in)

5.2.3.4 AC Overlay on AC/PC Pavement

The structural analysis for AC overlay on existing AC/PCC pavement design is similar to the procedure in bonded PCC overlay designs. The differences are in Steps 2 and 3:

Step 2. Determine the effective thickness D_{eff} of the existing AC/PCC pavement

The effective thickness D_{eff} of the existing AC/PCC pavement is determined by the Condition Survey Method:

$$D_{eff} = \left(D_{pccx} \times F_{jc} \times F_{dur}\right) + \left(\frac{D_{ac}}{2.0} \times F_{ac}\right) \tag{5.18}$$

where:

D_{pcc}	=	thickness of the existing PCC slab, mm (in)
F_{jc}	=	joint and cracks adjustment factor
F_{dur}	=	durability adjustment factor
D_{ac}	=	thickness of the existing AC surface, mm (in)
F_{ac}	=	quality factor of the existing AC surface.

Step 3. Determine the required overlay thickness (D_{ol})

The AC overlay thickness D_{ol} is calculated with Equation (5.15).

It should be noted that only AC material is used in future overlay analysis for new AC overlay designs, and the designer may indicate a mill-off depth on the existing AC layer and future AC overlays from the input.

5.3 FWD Backcalculation

For pavement rehabilitation projects a good understanding of the existing pavement in terms of the layered material properties and the subgrade condition is fundamental to the success of pavement designs. However, when such information is not available, it is also not economical nor practical for a highway agency to make extensive destructive tests for all the rehabilitation projects in the network. Non-destructive testing, such as with the Falling Weight Deflectometer (FWD), is a valuable tool to acquire the missing information.

Since the linkage between the OPAC pavement design method and FWD pavement evaluation has not been established at the present time, the backcalculation program in OPAC 2000 is only available for rigid pavement analysis. Falling Weight Deflectometer survey results can be used to estimate the subgrade reaction coefficient k, the elastic modulus Ec and the rupture modulus S'c of the existing PCC slab as well as the load transfer coefficient J by running the FWD Backcalculation subroutine in the OPAC 2000 system. The calculation procedures described here are organized separately for existing PCC pavements and for existing AC/PCC pavements. They are all based on the AASHTO Guide (Chapter 5, Part III).

5.3.1 Backcalculation for Existing PCC Pavements

1. Backcalculation of Modulus of Subgrade Reaction k

The backcalculation procedure for subgrade reaction k, or static modulus k, involves determining the deflection basin area (AREA), and the dense liquid radius of relative stiffness (l_k). AREA can be calculated with the following equation:

$$AREA = 6\,[1 + 2(d_{12}/d_0) + 2(d_{24}/d_0) + (d_{36}/d_0)] \tag{5.19}$$

where:

AREA = deflection basin area, mm^2 (in^2)

d_0 = maximum deflection at the centre of the loading plate, mm (in)

d_i = deflection at 30.5 cm (12 in), 61 cm (25 in), and 91.5 cm (36 in) from the plate centre, mm (in).

The dense liquid radius of relative stiffness l_k (mm (in)) can be determined with the following equation:

$$l_k = \left[\frac{\ln\left(\frac{36 - AREA}{1812.279}\right)}{-2.55934}\right]^{4.387009} \tag{5.20}$$

The dynamic modulus k_{dyn} (MPa/mm (psi/in)) of subgrade reaction is determined with the following equation:

$$k_{dyn} = \left(\frac{P}{8 d_0 l_k^{\,2}}\right)\left\{1 + \frac{1}{2\pi}\left[\ln\left(\frac{a}{2 l_k}\right) + \gamma - 1.25\right]\left(\frac{a}{l_k}\right)^2\right\} \tag{5.21}$$

where:

P = load plate pressure, kN (lbs)

d_0 = maximum deflection at the center of load, mm (in)

l_k	=	determined by Equation (5.20), mm (in)
a	=	load plate radius, mm (in)
γ	=	Euler's constant, 0.57721566490,

The static modulus k (MPa/mm (psi/in)) is estimated as a half of dynamic modulus k_{dyn}.

$$k = k_{dyn}/2 \tag{5.22}$$

2. Backcalculation of Elastic Modulus E_c

The elastic modulus E_c (MPa (psi)) is backcalculated as:

$$E_c = [12(1-\mu^2)k_{dyn}l_k^4]/D^3 \tag{5.23}$$

where:

μ	=	Poisson's ratio for concrete
k_{dyn}	=	determined by Equation (5.21), MPa/mm (psi/in)
l_k	=	determined by Equation (5.20), mm (in)
D	=	slab thickness, mm (in)

3. Backcalculation of Rupture Modulus S'_c

The rupture modulus S'_c (MPa (psi)) is backcalculated as:

$$S'_c = 43.5(E_c/10^6) + 488.5 \tag{5.24}$$

where:

E_c	=	determined by Equation (5.23), MPa (psi)

4. Backcalculation of Load Transfer Coefficient J

The load transfer coefficient J depends on the percentage load transfer ΔLT. ΔLT can be determined by measuring the deflection at the centre of the load plate, (place the load plate on one side of the joint) and at 300 mm from the centre using the following equation:

$$\Delta LT = 100 \times (\Delta_u / \Delta_l) \times B \tag{5.25}$$

where:

ΔLT	=	deflection load transfer, percent
Δ_{ul}	=	unloaded side deflection, mm (in)
Δ_l	=	loaded side deflection, mm (in)
B	=	slab bending correction factor

B is determined from the ratio of d_0 to d_{12} for typical centre slab deflection basin measurements, using the following equation:

$$B = d_{0\,centre} / d_{12\,centre} \tag{5.26}$$

For JPCP and JRCP, determine the load transfer coefficient J using the following guidelines:

ΔLT	J
>70 %	3.2
50 - 70 %	3.5
<50 %	4.0

For overlays designed on existing CRCP, J value is recommended to be between 2.2 and 2.6 [AASHTO 1993].

5.3.2 Backcalculation for Existing AC/PCC Pavements

1. Backcalculation of Modulus of Subgrade Reaction k

The backcalculation procedure for subgrade reaction k, or static modulus k, involves determining the deflection basin area $AREA_{pcc}$ and the dense liquid radius of relative stiffness (l_k). To determine $AREA_{pcc}$, the deflection of the slab at the center of load, $d_{o\,pcc}$, must first be modified with the following equation:

$$d_{o\,pcc} = d_0 - d_{0\,comp} \tag{5.27}$$

where:

d_0	=	maximum deflection at center of load, mm (in)
$d_{0\,comp}$	=	AC compression at center of load, mm (in)

The AC compression at the center of load can be determined as follows:

(1) When AC layer is removed:

$$d_{0\,comp} = 0 \quad (5.28)$$

(2) When AC and PCC layers are bonded:

$$d_{0\,comp} = -0.0000328 + 121.5006\,(D_{ac}/E_{ac})^{1.0798} \quad (5.29)$$

where:

D_{ac} = AC layer thickness, mm (in)

E_{ac} = elastic modulus of the AC layer, MPa (psi)

(3) When AC and PCC layers are unbonded:

$$d_{0\,comp} = -0.00002132 + 38.6872\,(D_{ac}/E_{ac})^{0.94551} \quad (5.30)$$

where:

D_{ac} and E_{ac} are described as above.

Then the deflection area AREApcc (mm (in)) of the slab can be calculated with following equation:

$$AREA_{pcc} = 6\,[1 + 2(d_{12}/d_{0\,pcc}) + 2(d_{24}/d_{0\,pcc}) + (d_{36}/d_{0\,pcc})] \quad (5.31)$$

where:

$d_{0\,pcc}$= PCC deflection in centre of loading plate that is the difference between surface deflection d_0 and AC compression $d_{0\,compress}$, mm (in)

d_i = deflection at 30.5 cm (12 in), 61 cm (24), and 91.4 cm (36 in) from plate centre, mm (in).

The dense liquid radius of relative stiffness l_k can be computed with the following equation:

$$l_k = \left[\frac{\ln\left(\dfrac{36 - AREA_{pcc}}{1812.279}\right)}{-2.55934}\right]^{4.387009} \quad (5.32)$$

where:

$AREA_{pcc}$ is determined by Equation (5.31).

With l_k from Equation (5.32), the dynamic modulus k_{dyn} of subgrade reaction and static modulus k can be determined using Equation (5.21) and Equation (5.22), respectively.

2. Backcalculation of Elastic Modulus E_c

The elastic modulus E_c is determined with the same equation as Equation (5.23):

$$E_c = [12(1- \mu^2)k_{dyn}l_k^4]/D^3 \tag{5.33}$$

where:

μ	=	Poisson's ratio of concrete
k_{dyn}	=	determined by Equation (5.21) with l_k determined by Equation (5.20)
l_k	=	determined by Equation (5.32)
D	=	thickness of existing slab, mm (in)

3. Backcalculation of Rupture Modulus S'_c

The rupture modulus S'_c (MPa (psi)) is backcalculated with the same equation as Equation (5.24):

$$S'_c = 43.5(E_c/10^6) + 488.5 \tag{5.34}$$

where:

E_c is determined by Equation (5.33), MPa (psi).

5.4 Sample Analysis

A four-lane portland cement concrete (PCC) pavement is under consideration in this example. Asphalt concrete is planned to be used as the future overlay material. An analysis period of 30 years is used. The structural-related inputs are given in Table 5.2.

Table 5.2 Sample Project Data of Rigid Pavement Analysis

Layers and Site Information		Other Design Criteria	
PCC slab	250 (mm)	Lane width	3.75 (m)
Granular Subbase	150 (mm)	Div'd/Und	Undivided
Future Overlay	50 (mm)	Initial performance index (P_0)	95
Mill-off depth before future overlay	25 (mm)	Performance index after future overlays (P_{future})	90
PCC slab elastic modulus (Ec)	29000 (MPa)	Minimum acceptable performance index (P_t)	50
Subbase elastic modulus (Esb)	140 (MPa)	Reliability (R)	0.9
PCC slab rupture (S'c)	4 (MPa)	S_0*	0.3
Roadbed Soil Strength (M_R)	42 (MPa)		
Subgrade Depth (Dsg)	1500 (mm)		
Load Transfer (J)	2.8		
Drainage	1.0		

* Combined standard error of the traffic prediction and performance prediction

The traffic load anticipated on the above pavement structure is 20,000 initial AADT increasing at a fixed rate of 3.5% per year. There is 10% of total trucks in the traffic flow, 40% of it is two and three axle trucks, 30% four axle trucks, 20% five axle trucks and 10% six and more axle trucks. The analysis period is 30 years. The foregoing traffic inputs translate into a yearly 80 kN equivalent single axle load (ESAL) of 362,400 in the first year and 762,236 by Year 30.

Applying the OPAC 2000 structural analysis procedure, the subgrade reaction k-value of the above pavement structure is 35.1. The predicted yearly pavement performance is given in Table 5.3.

Table 5.3 Sample Rigid Structural Analysis Results

1st Period		2nd Period	
Year	PCI	Year	PCI
0	95.0	17	90.0
1	93.1	18	87.7
2	91.0	19	85.3
3	88.8	20	82.8
4	86.6	21	80.2
5	84.2	22	77.5
6	81.7	23	74.8
7	79.1	24	72.0
8	76.4	25	69.0
9	73.6	26	66.0
10	70.7	27	62.9
11	67.6	28	59.8
12	64.5	29	56.5
13	61.3	30	53.2
14	57.9		
15	54.4		
16	50.8		

The table shows that the rigid pavement design alternative will have an initial life of 16 years with a 90% reliability. It requires a future overlay at Year 17. By the end of the 30 year analysis period the performance index will be about 53 PCI.

After the structural analysis is finished, all the feasible pavement design alternatives and the performance analysis results are entered into the economic analysis module for life cycle cost analysis.

CHAPTER 6 ECONOMIC ANALYSIS MODULE

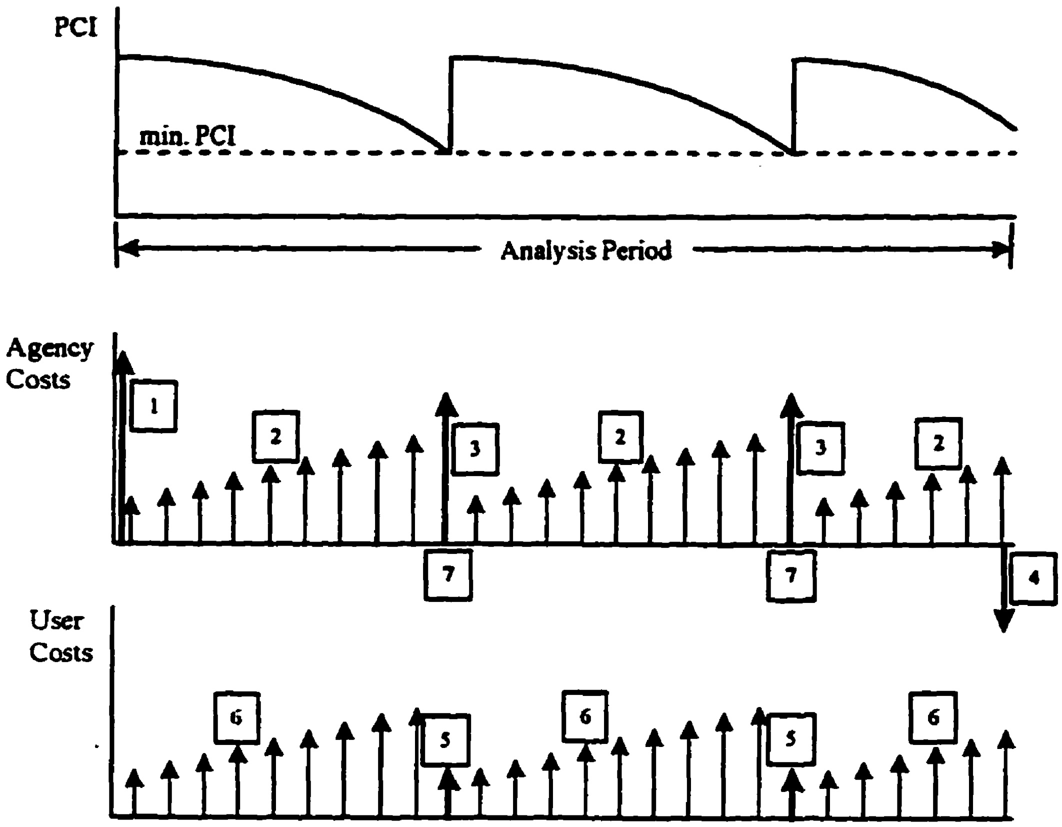
The economic analysis module in OPAC 2000 deals with two major types of costs: highway agency costs and road user costs. Both are updated and enhanced from the current OPAC [Kher 1975]. Within the analysis period agency costs include the initial pavement construction cost, maintenance cost, pavement rehabilitation cost and the residual value at the end of the analysis period. The agency cost stream is schematically displayed in the middle part of Figure 6.1. The downward arrow means that the residual value represents a cost recovery.

Road user costs (referred to as the user costs hereafter) includes the annual vehicle operating cost (VOC) and the traffic delay cost due to pavement rehabilitation (overlay) interruptions. Both types of cost are related to the pavement performance history as shown in the top part of Figure 6.1. In the user cost stream, VOC increases as the pavement deteriorates. When PCI reaches the minimum level and a pavement rehabilitation is triggered, the user delay cost is induced. Meanwhile, vehicle emission during the rehabilitation is also predicted to help assess the impact to the environment.

Figure 6.2 shows how these costs are interrelated and how each cost calculation is started. It also shows that all cost elements are added up at each year and discounted to the present worth with a discount rate specified by the pavement design engineer. The process is repeated for each year of the whole analysis period to obtain the total costs. The total cost calculation is performed for each design alternative, and finally, the design alternatives are ranked from the least total cost to the most expensive one in the output report.

6.1 Agency Costs

Agency costs considered in the OPAC 2000 economic analysis module include the initial construction cost, rehabilitation costs and maintenance costs. Initial construction cost (INC) and rehabilitation construction cost (RHC) are costs to build the traffic lanes, shoulders and other parts of the pavement. Maintenance costs (MC) include the routine maintenance cost and non-routine or one-time maintenance cost. Administration costs of the agency are not included in OPAC 2000, because they should not affect the choice of a design strategy.



Total Costs

- Agency Costs
  1. Initial Construction Cost
  2. Maintenance Cost
  3. Rehabilitation Cost
  4. Residual Values
- User Costs
  5. User Delay Cost
  6. Vehicle Operating Cost

(7. Emission, not in dollars)

Figure 6. 1 OPAC 2000 Cost Analysis

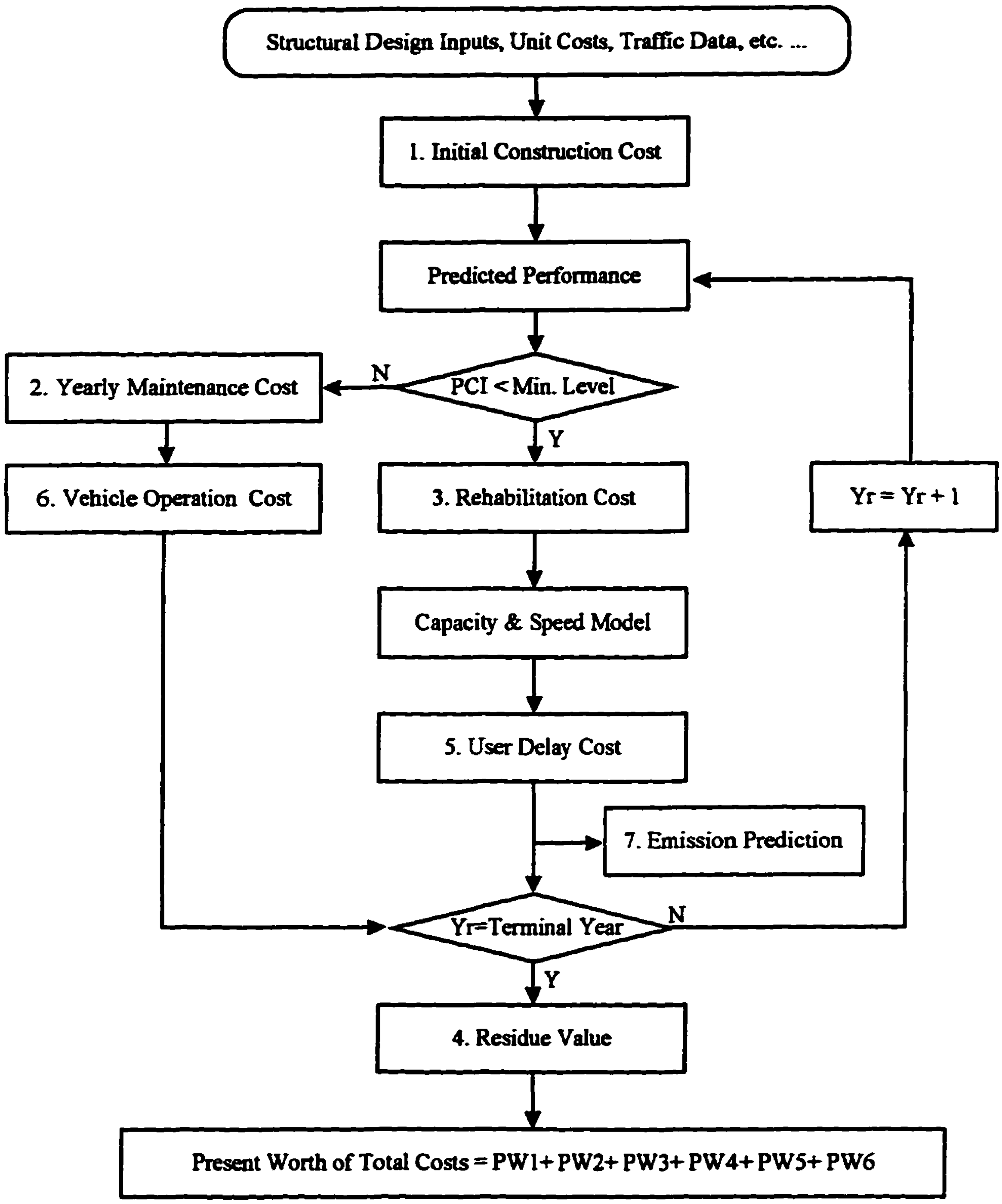


Figure 6.2 Structure of OPAC 2000 Economic Analysis Module

Since OPAC 2000 uses the present worth method in the economic analysis, the cost elements that happen in the future, such as rehabilitation construction cost, maintenance cost and the residual value, need to be discounted to the present time with a discount rate determined by the designer.

To increase the flexibility in application, OPAC 2000 is designed to accept various cost units such as unit cost by weight or by volume and lump sum cost. This is accomplished through two built-in libraries (database): Material Library and Maintenance Activity Library. The two libraries contain typical materials and their unit costs used in Ontario as well as typical maintenance activities with costs. For applications outside Ontario, the items in the libraries need to be updated according to the local situations.

6.1.1 Initial Construction Cost

Initial construction cost is the cost to build traffic lanes and shoulders. After the structural analysis the cross-sectional dimensions of the pavement structure are determined for each design alternative which include the lane width and the number of lanes, layer thickness, shoulder width and thickness, etc. The material quantity of each pavement layer and shoulders is determined based on this information. Unit costs of materials are from either the pre-edited material library or entered from the interface windows provided with the system. The product of the material quantity and the unit cost gives the initial construction cost. Initial construction cost is considered to occur at the beginning of the analysis period, therefore, it is already in the form of the present worth (PWINC).

6.1.2 Rehabilitation Cost

As with the initial construction cost, rehabilitation cost is the product of the material quantity and the unit cost, but it only involves the future overlay materials. The thickness of the future overlay is given by the designer in the input, and the timing of future overlays is determined through the structural analysis. Future rehabilitation construction costs needs to be discounted to the present time for its present worth (PWRHC), as expressed in the following equation:

$$PWRHC = \sum_{i} \frac{RHC_i}{(1+r)^i} \tag{6.1}$$

where:

PWRHC	=	present worth of total rehabilitation costs, \$/km;
RHC_i	=	rehabilitation cost at Year i, \$/km;
r	=	discount rate, specified by the user, and
i	=	number of years from the present time to each rehabilitation year.

6.1.3 Maintenance Cost

OPAC 2000 takes into account two types of maintenance costs. (1) The routine maintenance cost can take two forms of annual increase: a constant amount increase or a constant percentage increase; (2) The non-routine maintenance or one-time maintenance cost may take place at any year(s) specified by the user during the input process. For the routine maintenance cost calculation, the designer has to specify the base year maintenance cost and a growth rate so that OPAC 2000 can compute the annual costs in later years. For one-time maintenance costs, users have to enter the cost values and the corresponding years.

The present worth of total maintenance cost (PWMC) is the summation of yearly maintenance costs which include one-time maintenance costs in the specified year(s):

$$PWMC = \sum_i \frac{mc_i}{(1+r)^i} + \sum_j \frac{MC_j}{(1+r)^j} \tag{6.2}$$

where:

PWMC	=	present worth of total maintenance cost, \$/km;
mc_i	=	routine maintenance cost at Year i, \$/km;
MC_j	=	one-time maintenance cost at Year j, \$/km;
r	=	discount rate, and
i, j	=	number of years from the present time to each maintenance activity year.

6.1.4 Residual Value

The residual value (RSV) in OPAC 2000 refers to the terminal value plus the salvage value. The terminal value is based on the remaining serviceability of the pavement at the end of analysis period. It is a function of pavement condition index (PCI) and the last rehabilitation cost. The present worth of terminal value is determined as follows:

$$PWTMV = \frac{RHC_i}{(PCI_i - minPCI)} \times \frac{(PCI_t - minPCI)}{(1+r)^i} \qquad (6.3)$$

where:

PWTMV	=	the present worth of the pavement terminal value, \$/km;
RHC_i	=	the last time rehabilitation cost, \$/km;
PCI_i	=	PCI immediately after the last rehabilitation at Year i ;
PCI_t	=	PCI at the end of analysis period;
minPCI	=	minimum acceptable PCI specified by the user;
r	=	discount rate, and
i	=	number of years from the last rehabilitation to the present.

On the other hand, the salvage value is defined as the value of reusable materials in the existing pavement structure when the PCI reaches the minimum PCI level. It is a fraction of each layer cost (entered as a percentage) estimated by the designer based on his/her experience. The present worth of residual cost PWRSC can be calculated as:

$$PWRSV = \sum PWSLV + \sum PWTMV \qquad (6.4)$$

where:

PWTMV = present worth of the terminal value at the end of analysis period, \$/km

PWSLV = present worth of salvage values by the end of analysis period, \$/km

The residual cost from Equation (6.4) is in effect a negative cost, as it represents a returned value at the end of the analysis period.

6.2 User Costs

User costs considered in the pavement design period include the user delay cost and vehicle operation cost (VOC). User delay cost is induced by pavement rehabilitation constructions. Generally speaking pavement design alternatives with more future overlays will be associated with higher user delay cost. Vehicle operation cost refers to the increased user expenses on vehicles due to the deteriorated pavement condition. There are other types of user cost that may relate to pavement condition, e.g., accident cost. Because of the limitations in acquiring precise data that separates accidents due to the worsening of pavement condition from those due to human errors, accident cost is not included in the OPAC 2000 road user cost analysis.

Since all user costs happen in the future, they need to be converted to the present worth with a discount rate given by the designer. The method of the conversion is the same as in agency cost calculations.

6.2.1 User Delay Cost

The user delay cost calculation translates the time delay into cost by the value of time. The delay consists of two parts: (1) the *slowing delay* due to the reduced speed through the work zone on the pavement and (2) the *queuing delay* due to the congestion when the traffic demand exceeds the reduced capacity during the construction. To determine the slowing delay, two types of speeds, the normal speed and the reduced speed, have to be determined as described later in the speed model. Related to calculating the speeds are the normal highway capacity and the reduced capacity under construction, which are also used to determine the queuing delay. The normal and reduced capacity calculations are described in the capacity model.

6.2.1.1 Traffic Control Plans

There are various combinations of traffic handling methods that can be used during the pavement rehabilitation. In OPAC 2000 eight traffic control plans are used for two-lane highways, multilane undivided highways and multilane divided highways. The schematic layouts are shown in Figures 6.3 and 6.4, in which the shaded areas represent the paving work zones. Table 6.1 is the relation between highway types and the traffic control plans:

Table 6.1 Highway Types and Traffic Control Plans

Highway Type	Undivided	Divided
2-Lane	Plans 1 & 2	N.A.
4-Lane	Plan 3	Plan 5
6-Lane	Plan 4	Plans 6 & 7
8-Lane	N.A.	Plan 8

The following procedures of calculating different types of delays are organized by the above traffic plans.

6.2.1.2 Slowing Delay

The slowing delay is evaluated as the difference between the longer travel time during construction and the normal travel time without construction, with the following equation:

$$D_j = \left(\frac{1.5}{Vr_j} - \frac{1.5}{Vn_j}\right) \tag{6.5}$$

where:

D_j	=	slowing delay due to low speed with traffic control Plan j (j = 2 to 8)[15], hour;
1.5	=	assumed length of work zone, km;
Vr_j	=	reduced speed with traffic control Plan j, km/h, and
Vn_j	=	normal speed corresponding to the reduced speed with traffic control Plan j.

Vr_j and Vn_j in Equation (6.5) are determined through the speed model, as subsequently described.

[15] Note that for j = 1, this is similar to a signalized intersection, as subsequently discussed.

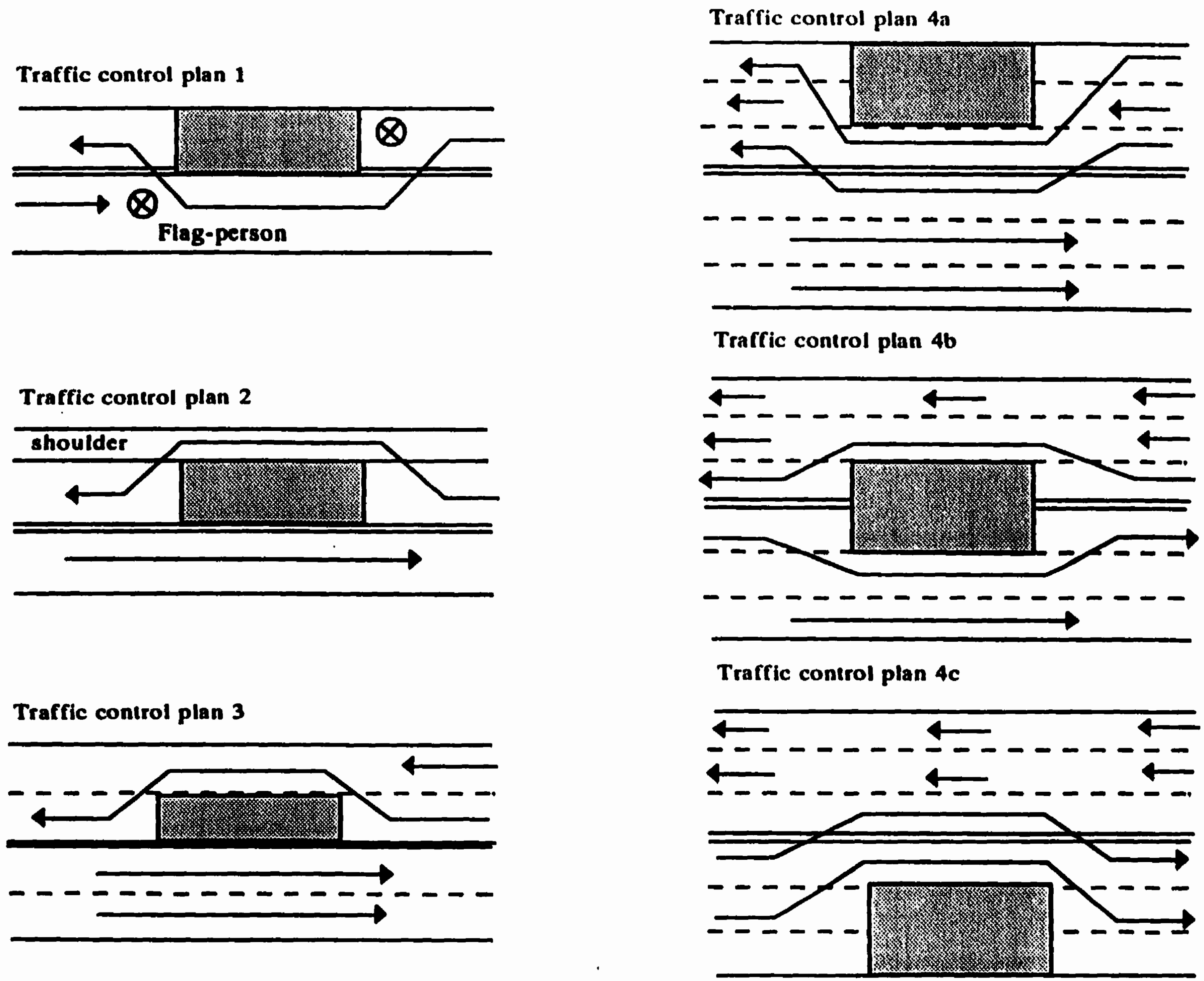


Figure 6.3 Traffic Control Plans 1 - 4

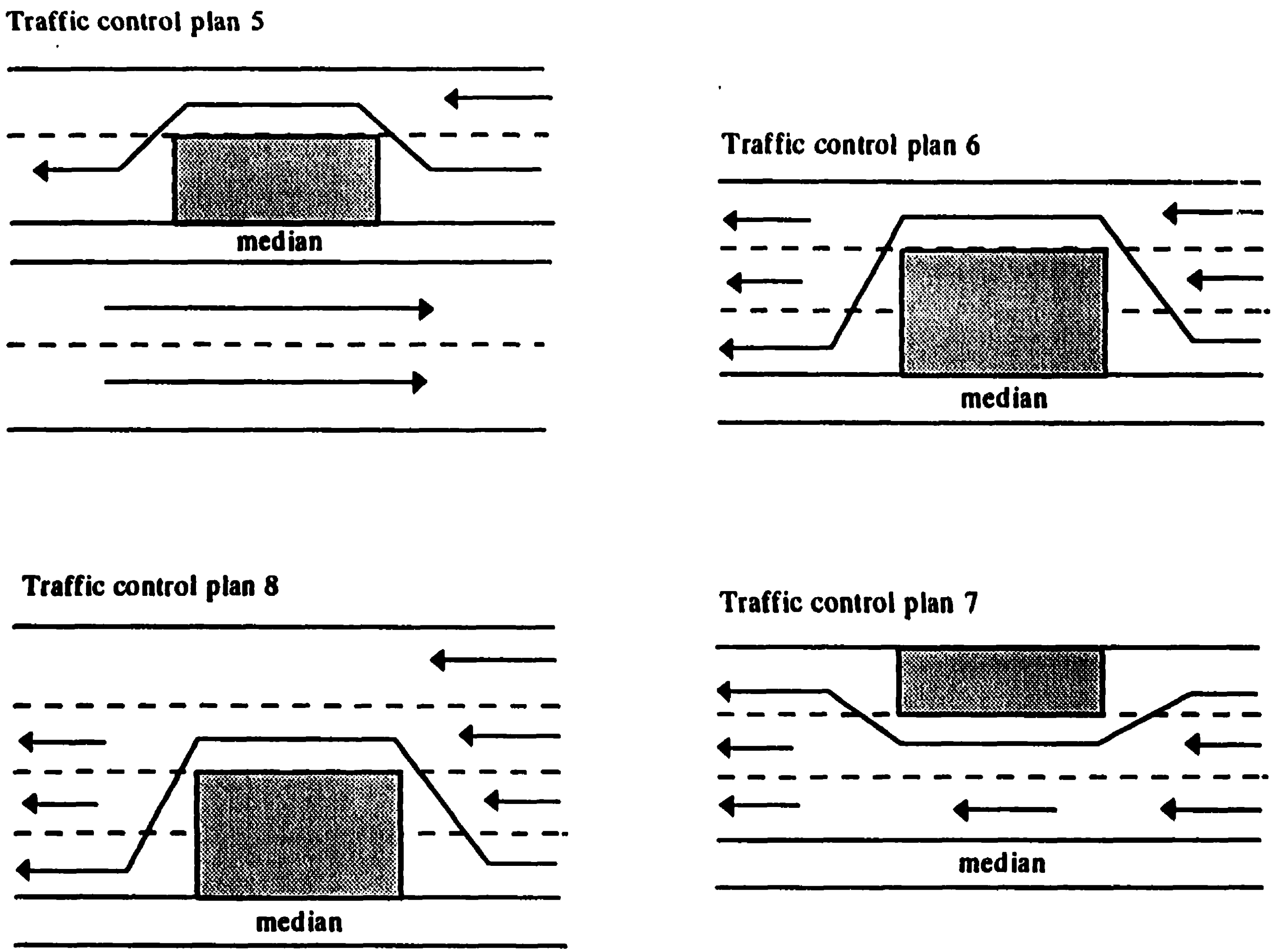


Figure 6.4 Traffic Control Plans 5 - 8

6.2.1.3 Queuing Delay

When the traffic demand exceeds the capacity, queuing delays (Dq) occur around the pavement work zone. The queuing delay is calculated in terms of the portion in a unit time (one hour) which depends on the selected traffic control plan. In quantitative terms, it is the ratio of the difference between the numbers of the arriving vehicles (ARR) and the leaving vehicles (LEA) to the reduced capacity (CAPr):

$$Dq = (ARR - LEA)/CAPr$$

The number of arriving vehicles in one hour is equal to the hourly volume HV, while the number of leaving vehicles refers to the vehicles passed through in one hour, which is equal to the work zone capacity, CAPr. Assuming equal delays for all arriving vehicles, during the period of one hour the queuing delay can be roughly estimated as:

$$Dq_j = (HV - CAPr_j)/CAPr_j \tag{6.6}$$

where:

Dq_j = queuing delay in one hour with traffic control Plan j, hour;

$CAPr_j$ = reduced capacity with traffic control Plan j, vph,

HV = two-way hourly volume where Plan 4 is applied and one-way hourly volume where Plans 2, 3, 5, 6, 7 and 8 are applied, vph.

Equation (6.6) indicates that when HV is less than or equal to $CAPr_j$, there is no queuing delay, and that the queuing delay occurs when HV is greater than $CAPr_j$.

The traffic demand is measured in terms of the hourly volume (HV), which is a proportion of average annual daily traffic (AADT). Changing with hours and seasons, HV in working hours in summer in Ontario can be estimated approximately with the following equation:

$$HV = 1.2 \times DF \times AADT \times HF \tag{6.7}$$

where:

1.2 = average summer factor [Karan 1974],

DF = directional split factor,

AADT = average annual daily traffic, vehicles per day, and

HF = hourly factor, 0.125 for two-lane highways and 0.07 for other highways.

For two-lane highways with a flagperson control, i.e., Plan 1, the delay equation for a signalized intersection presented in the 1994 HCM (Highway Capacity Manual) is used to simulate the situation:

$$D_1 = \{0.38C(1-g/C)^2/[1-(g/C)\mathrm{Min}(X, 1.0)]\}DAF + 173X^2\{(X-1) + [(X-1)^2 + mX/c]^{0.5}\}$$

where:

D_1 = stopping delay, sec/veh;

DAF = delay adjustment factor for quality of progression and control type;

X = v/c ratio for one group;

C = cycle length, sec;

c = capacity of lane group, vph;

g = effective green time for lane group, sec; and

m = an incremental delay calibration term representing the effect of arrival type and degree of platooning.

For simplification, assuming DAF = 1 and m = 16 and relaxing X, the above delay equation is rearranged as:

$$D_1 = \{0.38\ C(1-g/C)^2/[1-(g/C)X]\}/3600 + 173X^2\{(X-1) + [(X-1)^2 + 16X/c]^{0.5}\}/3600 \qquad (6.8)$$

where:

D_1 = average delay time under flag-person control, hour/veh;

c = reduced capacity ($CAPr_1$) under traffic control Plan 1, vph;

X = v/c ratio, v is two-way hourly volume HV (vph);

g = green time, sec; and

C = cycle length, sec.

Since the value of "g/C" ratio affects the capacity which then controls delay time, the green time and the cycle length are calculated at different traffic levels. The suggested green times and cycle lengths with the least delay at different traffic levels in terms of AADT are determined and listed in Table 6.2.

Table 6.2 Green Time and Cycle Length at Different Traffic Levels

AADT	Green Time (s)	Cycle length (s)
<3500	100	400
3500 - 4000	150	500
4000 - 6500	250	700
6500 - 7000	300	800
7000 - 7500	350	900
7500 - 8000	400	1000
8000 - 8500	450	1100
8500 - 9000	500	1200
9000 - 9500	570	1340
9500 - 10000	610	1420

With the assumption that the vehicle occupancy is one person per vehicle, and commodity delay costs are ignored, the user delay cost with traffic control Plan j can be calculated as follows:

$$DL_j = (D_j + Dq_j) \times JD_j \times HV \times JT_j \qquad (6.9)$$

where:

DL_j = user delays with traffic control Plan j, hour

D_j = delays due to low through speed with traffic control Plan j, hour;

Dq_j = queuing delays with traffic control Plan j (j = 2 to 8), hour;

JD_j = job duration, hour;

HV	=	two-way hourly volume, where: j = 1 or j = 4, and one-way hourly volume, where: j = 2, 3, 5, 6, 7, 8, vehicle (person) per hour;
JT_j	=	number of job times with traffic control plan j[16], and they are assigned as:

JT_1	=	2,	JT_2	=	2
JT_3	=	4,	JT_4	=	3
JT_5	=	4,	JT_6	=	2
JT_7	=	2,	JT_8	=	4

The delay cost on the one kilometer length construction[17] at Year i (DLC_i) is the product of the length of delay and the time value which equals to the hourly wage rate at Year i:

$$DLC_i = DL_i \times WG_i \tag{6.10}$$

where:

DLC_i	=	delay cost at Year i, $/km;
DL_i	=	average delays at Year i, hour, and
WG_i	=	hourly wage rate at Year i, $/hour.

The review of travel time values indicated that bi-weekly income levels in Ontario are most likely to fall into the range from $1,750 to $2,500 [Kazakov 1993]. For simplicity, a bi-weekly salary level of $2,000 is used as the basis of computing the user delay costs. Therefore, the hourly wage rate of $25 is then recommended as the value of travel time for people involved in the traffic fleet.

For six-lane undivided highways delays due to construction on all six lanes should be the combination of the delays under both Traffic Control Plan 6 and Traffic Control Plan 7:

$$DLC_i = (DL_6 + DL_7) \times WG_i \tag{6.11}$$

where:

DLC_i	=	delay cost at Year i, $/km;

[16] This is the number of times that a rehabilitation job has to cover the same section of highway, using a single lane pavement work zone or double lane work zone, as indicated by the shaded areas in Figures 6.3 and 6.4.

[17] Section 6.2.1.2 indicated a work zone length of 1.5 km. The length of construction within that work zone is assumed to be one km.

DL_6 = average delays at Year i with traffic control Plan 6, hour;

DL_7 = average delays at Year i with traffic control Plan 7, hour, and

WG_i = hourly wage rate at Year i, \$/hour.

The following equation (6.12) evaluates the present worth of traffic delay costs (PWDLC) during the future overlays of each pavement design alternative.

$$PWDLC = \sum_i \frac{DLC_i}{(1+r)^i} \tag{6.12}$$

where:

PWDLC= present worth of total delay costs, \$/km;

DLC_i = delay cost at Year i, \$/km;

r = discount rate, and

i = number of years from the present time to when the delay happens, year.

6.2.1.4 The Capacity Model

The capacity of highways is calculated under two situations. The normal capacity refers to the road capacity without construction, and the reduced capacity is that under the condition with closure of a certain number of lanes during the construction. The two types of capacities are calculated based on the Highway Capacity Manual (HCM) [TRB 1985, TRB 1994]. The normal capacity is a function of the cross-sectional characteristics of the highways which are divided into two-lane highways, multilane undivided highways and multilane divided highways. The reduced capacity varies with the traffic alteration method (traffic control plans).

1. Two-Lane Highways

The normal capacity ($CAPn_1$) is used when traffic control Plan 1 is selected, and the normal capacity ($CAPn_2$) is used when traffic control Plan 2 is selected. The normal capacity for two-lane highways, either $CAPn_1$ or $CAPn_2$, in vehicles per hour per lane (vphpl), can be determined by the following equation:

$$CAPn_1 = CAPn_2 = 1400 \times 0.72 \times F_{wn} \tag{6.13}$$

where:

$CAPn_{1,2}$= one lane normal capacity for two-lane highways, vphpl;

1400 = passenger cars per hour per lane under ideal conditions, pcphpl;

0.72 = adjustment factor for the presence of heavy vehicles in the traffic stream, and

F_{wn} = adjustment factor for narrow lanes and restricted shoulder widths.

The following functions for calculating F_{wn} are based on Table 8-5 in HCM (with R squares greater than 0.998):

In the case that lane width is 3.75 m:

$$F_{wn} = 0.71285 + 0.20935\ SHD - 0.\ 02104\ SHD^2 \tag{6.14}$$

In the case that lane width is 3.5 m:

$$F_{wn} = 0.67295 + 0.19016\ SHD - 0.\ 016662\ SHD^2 \tag{6.15}$$

In the case that lane width is 3.25 m:

$$F_{wn} = 0.62588 + 0.17889\ SHD - 0.\ 015654\ SHD^2 \tag{6.16}$$

In the case that lane width is 3.0 m:

$$F_{wn} = 0.56485 + 0.17525\ SHD - 0.\ 020156\ SHD^2 \tag{6.17}$$

In the case that lane width is 2.75 m:

$$F_{wn} = 0.49142 + 0.15413\ SHD - 0.\ 020156\ SHD^2 \tag{6.18}$$

where:

SHD = pavement shoulder width, m

The reduced capacity ($CAPr_{1,2}$) is calculated under two different conditions. When the shoulder is too narrow (less than 3 m) to pass vehicles, only one traffic lane can be opened to the traffic and the other lane is closed for construction. In this case, the method of a flag person control, i.e., traffic control Plan 1, has to be used. With the flag person control, the reduced capacity $CAPr_1$ is controlled by the green time g and the cycle length C as expressed by the following equation:

$$CAPr_1 = CAPn_1 \times \frac{g}{C} \tag{6.19}$$

where:

$CAPr_1$ = the reduced capacity using traffic control Plan 1, vphpl;

$CAPn_1$ = the normal capacity using traffic control Plan 1, vphpl;

g = green time from Table 6.2, second, and

C = cycle length from Table 6.2, second.

In the case of wide shoulders (equal or greater than 3 m defined in OPAC 2000), the shoulders are capable of carrying traffic through. To capture the restricted driving condition on the shoulder, an adjustment factor of $F_{wn} = 0.565$ is needed in estimating the shoulder capacity $CAPr_2$.

$$CAPr_2 = 1400 \times 0.72 \times F_{wn} = 1400 \times 0.72 \times 0.565 = 570 \qquad (6.20)$$

where:

$CAPr_2$ = the reduced capacity using traffic control Plan 2, vphpl;

1400 = passenger cars per hour per lane under an ideal condition, pcphpl;

0.72 = adjustment factor for the presence of heavy vehicles in the traffic stream, and

F_{wn} = adjustment factor for narrow lanes and restricted shoulder widths, 0.565 is determined using Equation (6.17).

2. Multilane Undivided Highways

The traffic control Plans 3 and 4 are used for 4-lane undivided and 6-lane undivided highways, respectively. The capacity calculations are based on the HCM service flow rate V_p calculation procedure for multilane undivided highways.

For Plan 3:

$$CAP_3 = Vp = HV/(n \times PHF \times F_{hv}) = HV/(n \times 0.88 \times 0.72) \qquad (6.21)$$

where:

V_p = service flow rate, passenger cars per hour per lane, pcphpl. The maximum V_p is 2200 pcphpl,

HV = one-way hourly volume, vph,

n = number of lanes opened to the traffic, use n = 2 for normal capacity and n = 1 for reduced capacity,

PHF = peak hour factor, assumed to be 0.88,

F_{hv} = heavy-vehicle adjustment factor, assumed to be 0.72

For Plan 4:

$$CAP_4 = V_p = HV/(n \times PHF \times F_{hv}) = HV/(n \times 0.88 \times 0.72) \qquad (6.22)$$

where:

V_p = service flow rate, passenger cars per hour per lane, pcphpl. The maximum V_p is 2200 pcphpl,

HV = two-way hourly volume, vph,

n = number of lanes opened to the traffic, use n = 6 for normal capacity and n = 4 for reduced capacity,

PHF = peak hour factor, assumed to be 0.88,

F_{hv} = heavy-vehicle adjustment factor, assumed to be 0.72

3. Multilane Divided Highways

The traffic control Plan 5 is used for 4-lane divided highways. Plans 6 and 7 are used for 6-lane divided highways. Plan 8 is used for 8-lane divided highways. The normal capacity ($CAPn_5$, $CAPn_6$, $CAPn_7$ and $CAPn_8$) of divided highways is estimated by adjusting the ideal capacity with a factor F_{hv} = 0.72 which counts for the presence of heavy vehicles:

$$CAPn_5\,,\ CAPn_6\,,\ CAPn_7\,,\ CAPn_8 = 2000 \times F_{hv} = 2000 \times 0.72 = 1440 \qquad (6.23)$$

where:

$CAPn_5$, $CAPn_6$, $CAPn_7$ and $CAPn_8$ are normal capacity of the multilane divided highways, vphpl;

2000 = capacity under ideal conditions, vphpl

F_{hv} = adjustment factor for the presence of heavy vehicles in the traffic stream, assumed to be 0.72.

The reduced capacities of $CAPr_5$ to $CAPr_8$ corresponding to traffic control Plans 5 to 8 are as follows:

$$CAPr_5 = CAPr_6 = 1030 \text{ vehicles per hour per lane} \quad (6.24)$$

$$CAPr_7 = CAPr_8 = 2600 \text{ vehicles per hour per two lanes} \quad (6.25)$$

The procedures of calculating the capacities are summarized in Table 6.3.

6.2.1.5 The Speed Model

1. Two-Lane Highways

Vehicle speeds on a two-lane highway normally depend on the road geometry, the length of passing zone and the traffic volume. In order to determine the speed value using Table 8-1 from the 1985 HCM, some simplifications have to be made. First, 20 percent of length is assumed as no passing zone. Next, the speed values on the level terrain and on the rolling terrain are averaged. After the two simplifications, vehicle speed can be evaluated by the ratio of hourly traffic volume to the capacity. The resulting functions are as follows:

$$Vn_2 = 99.322 - 71.047(HV/CAPn_2) + 100.14(HV/CAPn_2)^2 - 61.622(HV/CAPn_2)^3 \quad (6.26)$$

$$Vr_2 = 94.584 - 60.406(HV/CAPr_2) + 90.133(HV/CAPr_2)^2 - 58.505(HV/CAPr_2)^3$$

or

$$Vr_2 = 94.584 - 60.406(HV/570) + 90.133(HV/570)^2 - 58.505(HV/570)^3 \quad (6.27)$$

where:

Vn_2 = normal speeds on two-lane highways, km/h.

Vr_2 = reduced speed on two-lane highways with traffic control Plan 2, km/h,

HV = hourly volume, vphpl,

$CAPn_2$ = normal capacity determine by Equation (6.13),

$CAPr_2$ = reduced capacity of 570 vph on two-lane highways using Plan 2.

Table 6.3 Summary of Capacity Calculations by Highway Types

Highway Type	Normal Capacity	Reduced Capacity
2-lane	$CAPn_1 = CAPn_2 =$ $1400 \times 0.72 \times F_{wn}$ vphpl	$CAPr_1 = CAPn_1 \times g/C$ vphpl $CAPr_2 = 570$ vphpl (shoulder capacity)
Multilane Undivided	$CAPn_3 = V_p =$ $HV/(2 \times 0.88 \times 0.72)$ vphpl $CAPn_4 = V_p =$ $HV/(6 \times 0.88 \times 0.72)$ vphpl	$CAPr_3 = V_p =$ $HV/(1 \times 0.88 \times 0.72)$ vphpl $CAPr_4 = V_p =$ $HV/(4 \times 0.88 \times 0.72)$ vphpl
Multilane Divided	$CAPn_5$, $CAPn_6$, $CAPn_7$, $CAPn_8$ = 1440 vphpl	$CAPr_5 = CAPr_6 = 1030$ vph per lane $CAPr_7 = CAPr_8 = 2600$ vph per 2 lanes

2. Multilane Undivided Highways

Based on Figure 7-4 in the 1994 HCM, vehicle speeds on a multilane highway are influenced by not only the service flow rate V_p but also the free-flow speed determined by lane widths, access points, etc. The free-flow speed FFS is evaluated by the following equation:

$$FFS = (FFSi - F_m - F_{lw} - F_{lc} - F_a) \times 1.609 = (60 - 1.6 - F_{lw} - F_{lc} - 2.5) \times 1.609 \qquad (6.28)$$

where:

FFS	=	estimated free-flow speed (km/h)
FFSi	=	60 mph, estimated free-flow speed under ideal conditions
F_m	=	adjustment for median type, 1.6 mph assumed, based on Table 7-2 in HCM.
F_{lw}	=	adjustment for lane width (ft), based on Table 7-3 in HCM.
F_{lc}	=	adjustment for lateral clearance (ft), based on Table 7-4 in HCM.
F_a	=	adjustment for access points, 2.5 mph assumed, based on Table 7-5 in HCM.

1.609 = conversion coefficient from Imperial to SI units.

According to HCM (Table 7-3), lane width adjustment factor F_{lw} (mile/h) can be calculated using the following equation:

$$F_{lw} = 207.6 - 34.1LW + 1.4LW^2 \tag{6.29}$$

where:

LW = lane width, feet.

For normal conditions (without construction), the lateral clearance adjustment factor F_{lc} can be calculated using the following equation (based on Table 7-4 in HCM):

For 4-lane highways:

$$F_{lc} = 5.4 + 0.3208\ SHD_{both} - 1.2036\ SHD_{both}^2 + 0.392968\ SHD_{both}^3 - 0.05499\ SHD_{both}^4 + 0.0035807\ SHD_{both}^5 - 0.000089\ SHD_{both}^6 \tag{6.30}$$

where:

SHD_{both} = lateral clearance (total width of both shoulders), feet.

For 6-lane highways:

$$F_{lc} = 3.9 + 0.1333\ SHD_{both} - 0.67507\ SHD_{both}^2 + 0.22109\ SHD_{both}^3 - 0.031033\ SHD_{both}^4 + 0.00202\ SHD_{both}^5 - 0.00005\ SHD_{both}^6 \tag{6.31}$$

where:

SHD_{both} = same as above

For restricted conditions (under construction), lane width adjustment factor F_{lw} is assumed to be 6.6 mile/h, and lateral clearance adjustment factor F_{lc} is set as 5.4 and 3.9 mile/h for four-lane highways and six-lane highways, respectively. Free flow speed (FFS) can be expressed as:

For 4-lane highways:

$$FFS = (60 - 1.6 - 6.6 - 5.4 - 2.5) \times 1.609 = 70.635\ (km/h) \tag{6.32}$$

For 6-lane highways:

$$FFS = (60 - 1.6 - 6.6 - 3.9 - 2.5) \times 1.609 = 73.084\ (km/h) \tag{6.33}$$

Knowing the service flow rate V_p, as described in the capacity model, and the free-flow speed FFS from Equation (6.28), the normal speeds (Vn_3 and Vn_4) and reduced speeds (Vr_3 and Vr_4) can be derived based on HCM (Figure 7-4) [TRB 1985] as follows:

$$Vn, Vr = FFS - f(V_p) \tag{6.34}$$

where:

$f(V_p)$	=	$6.5333\times10^{-5}\, V_p + 3.7893\times10^{-7}\, V_p^{\,2} - 1.3311\times10^{-9}\, V_p^{\,3} + 8.6264\times10^{-13}\, V_p^{\,4}$,
V_p	=	service flow rate, pcphpl

Substituting V_p with the normal capacities ($CAPn_3$ and $CAPn_4$) and the reduced capacities ($CAPr_3$ and $CAPr_4$) as defined in the Capacity Model, Equation (6.32) can be expressed as:

Normal speed on 4-lane highways:

$$\begin{aligned} Vn_3 = FFS - [&6.5333\times10^{-5}\,(HV/1.2672) + 3.7893\times10^{-7}\,(HV/1.2672)^2 \\ &- 1.3311\times10^{-9}\,(HV/1.2672)^3 + 8.6264\times10^{-13}\,(HV/1.2672)^4] \end{aligned} \tag{6.35}$$

where:

Vn_3	=	normal speed on four-lane highways, km/h,
FFS	=	free flow speed on four-lane highways determined by Equations (6.28), (6.29) and (6.30), km/h,
HV	=	one-way hourly volume, vph

Normal speed on 6-lane highways:

$$\begin{aligned} Vn_4 = FFS - [&6.5333\times10^{-5}\,(HV/3.8016) + 3.7893\times10^{-7}\,(HV/3.8016)^2 \\ &- 1.3311\times10^{-9}\,(HV/3.8016)^3 + 8.6264\times10^{-13}\,(HV/3.8016)^4] \end{aligned} \tag{6.36}$$

where:

Vn_4	=	normal speed on six-lane highways, km/h
FFS	=	free flow speed on six-lane highways determined by Equations (6.28), (6.29) and (6.31), km/h
HV	=	one-way hourly volume, vph

Reduced speed on 4-lane highways

$$Vr_3 = FFS - [6.5333\times10^{-5} (HV/0.6336) + 3.7893\times10^{-7} (HV/0.6336)^2 - 1.3311\times10^{-9} (HV/0.6336)^3 + 8.6264\times10^{-13} (HV/0.6336)^4] \quad (6.37)$$

where:

Vr_3 = normal speed on four-lane highways, km/h

FFS = free flow speed on four-lane highways determined by Equations (6.28), (6.29) and (6.32), km/h

HV = one-way hourly volume, vph

Reduced speed on 6-lane highways

$$Vr_4 = FFS - [6.5333\times10^{-5} (HV/2.5344) + 3.7893\times10^{-7} (HV/2.5344)^2 - 1.3311\times10^{-9} (HV/2.5344)^3 + 8.6264\times10^{-13} (HV/2.5344)^4] \quad (6.38)$$

where:

Vr_4 = normal speed on six-lane highways, km/h

FFS = free flow speed on six-lane highways determined by Equations (6.28), (6.29) and (6.33), km/h

HV = one-way hourly volume, vph

3. Multilane Divided Highways

The normal speed and reduced speed on a divided highway can be determined using the speed-flow relationship displayed in Figure 3-4 in the 1985 HCM. Knowing hourly volumes and capacities, or the v/c ratio, the normal speed can be determined using the curve of 70 mph design speed in the figure. The fitted results are as follows:

Normal speed Vn_5 (km/h) on 4-lane freeways

$$Vn_5 = 96.54 - 15.6936[HV/(2\times1440)]^{1.315435} - 32.5764[HV/(2\times1440)]^{15.7011} \quad (6.39)$$

Normal speeds Vn_6 and Vn_7 (km/h) on 6-lane freeways:

$$Vn_6 = Vn_7 = 97.345 - 18.6228[HV/(3\times1440)]^{1.9478} - 30.4522[HV/(3\times1440)]^{18.9453} \quad (6.40)$$

Normal speeds Vn_8 (km/h) on 8-lane freeways:

$$Vn_8 = 97.505 - 19.07[HV/(4\times1440)]^{2.4} - 30.1650[HV/(4\times1440)]^{16.0436} \quad (6.41)$$

where:

HV = one-way hourly volume, vph

The reduced speeds Vr_5, Vr_6, Vr_7 and Vr_8 (km/h), under traffic control Plans 5, 6, 7 and 8, can be approximated to follow the speed curve with 60 mph design speed in the figure. The fitted results are as follows:

$$Vr_5, Vr_6 = 89.2995 - 18.5384(HV/1030) - 22.4906(HV/1030)^{7.233} \quad (6.42)$$

$$Vr_7, Vr_8 = 89.2995 - 18.5384(HV/2600) - 22.4906(HV/2600)^{7.233} \quad (6.43)$$

where:

HV = one-way hourly volume, vph

In the case that v/c ratio is greater than one, the reduced speeds (km/h) are determined by the following equations:

$$Vr_5, Vr_6 = 209.17 - 160.9(HV/1030) \quad \text{when } 1 < (HV/1030) \le 1.2 \quad (6.44)$$

$$Vr_7, Vr_8 = 209.17 - 160.9(HV/2600) \quad \text{when } 1 < (HV/2600) \le 1.2 \quad (6.45)$$

$$Vr_5, Vr_6 = 16.09 \quad \text{when } (HV/1030) > 1.2 \quad (6.46)$$

$$Vr_7, Vr_8 = 16.09 \quad \text{when } (HV/2600) > 1.2 \quad (6.47)$$

where:

HV = one-way hourly volume, vph

The procedures used in the Speed Model are summarized in Table 6.4.

Table 6.4 Summary of Speed Calculations

Highway Type	Normal Speed (km/h)	Reduced Speed (km/h)
Two-lane	Vn_2 = f(HV/CAPn2) (6.26)	Vr_2 = f(HV) (6.27)
Multilane undivided	4-lane: Vn_3 = f(FFS, HV) (6.35) 6-lane: Vn_4 = f(FFS, HV) (6.36)	4-lane: Vr_3 = f(FFS, HV) (6.37) 6-lane: Vr_4 = f(FFS, HV) (6.38)
Multilane divided	4-lane: Vn_5 = f(HV) (6.39) 6-lane: $Vn_6 = Vn_7$ = f(HV) (6.40) 8-lane: Vn_8 = f(HV) (6.41)	4-lane: Vr_5 = f(HV) (6.42) 6-lane: $Vr_6 = Vr_5$ (6.42) 8-lane: Vr_7 = f(HV) (6.43) $Vr_8 = Vr_7$ (6.43)

6.2.2 Vehicle Operating Cost

Vehicle operating costs (VOC) in OPAC 2000 are calculated by a VOC model. VOC is a function of pavement performance and vehicle type. The performance predictions for both flexible pavements and rigid pavements have been described earlier in the structural analysis modules. For performing the VOC calculation the output of the performance index, PCI or PSI, needs to be converted into the International Roughness Index (IRI).

6.2.2.1 Converting PSI (PCI) Into IRI

The calculated PSI (for rigid pavements) or PCI (for flexible pavements) from the structural analysis is converted into IRI (unit: m/km) by the following equation [Paterson 1986]:

$$IRI = -5.555555 Ln(0.2PSI) \qquad (6.48)$$

To use the above relationship for flexible pavement designs, the predicted pavement performance in PCI needs to be converted into PSI by a factor of 1/20. The resulting IRI is used in the vehicle operating cost model in the economic analysis module.

6.2.2.2 VOC Calculation for Different Types of Vehicles

The vehicle operating cost (VOC) model in OPAC 2000 is used to calculate the increased VOC due to the increase of pavement roughness in terms of IRI. Because the pavement roughness (IRI) is used as a common platform, no differentiation is made for the VOC calculation for different pavement types.

In the calculating procedure vehicles are divided into three groups: Group A is for cars, Group B consists of 2-axle and 3-axle trucks and Group C includes trucks with 4 and more axles. In the software package Group A is determined by subtracting the truck portion (truck percent) from AADT. Group B consists of the first class in Table 4.6, or Classes 4, 5 and 6 in Table 4.7 (FHWA vehicle classes). Group C includes the vehicles of the remaining three classes in Table 4.6, or Classes 7 to 13 in Table 4.7.

The relationship between VOC and IRI by different vehicle groups is provided by Ontario VOC model version 3.0 [MTO 1993]. The extra VOC as a function of IRI for the three vehicle groups can be expressed by the following fitted equations (R squares are greater than 0.998):

$$VOCA\_IRI = 1.2542\ IRI + 0.42754\ IRI^2 \tag{6.49}$$

$$VOCB\_IRI = 22.604\ IRI + 1.4410\ IRI^2 \tag{6.50}$$

$$VOCC\_IRI = 18.545\ IRI + 1.6223\ IRI^2 \tag{6.51}$$

where:

VOCA_IRI, VOCB_IRI and VOCC_IRI are extra VOC ($/1000 veh-km) for Groups A, B and C, respectively.

IRI = International Roughness Index, m/km

For given traffic distributions, VOC in Year i for the three groups can be calculated by the following equations:

$$VOCA_i = DAYS \times AADT_i \times TYA \times VOCA\_IRI_i\ /1000 \tag{6.52}$$

$$VOCB_i = DAYS \times AADT_i \times TYB \times VOCB\_IRI_i\ /1000 \tag{6.53}$$

$$VOCC_i = DAYS \times AADT_i \times TYC \times VOCC\_IRI_i\ /1000 \tag{6.54}$$

where:

$VOCA_i$	=	VOC at Year i for Group A, \$/km;
$VOCB_i$	=	VOC at Year i for Group B, \$/km;
$VOCC_i$	=	VOC at Year i for Group C, \$/km;
$AADT_i$	=	average annual daily traffic at year i, vehicle per day;
TYA	=	proportion of Group A in AADT;
TYB	=	proportion of Group B in AADT;
TYC	=	proportion of Group C in AADT;
$VOCA\_IRI_i$	=	extra VOC at Year i for Group A determined by Equation (6.49), \$/1000 veh-km;
$VOCB\_IRI_i$	=	extra VOC at Year i for Group B determined by Equation (6.50), \$/1000 veh-km;
$VOCC\_IRI_i$	=	extra VOC at Year i for Group C determined by Equation (6.51), \$/1000 veh-km; and
DAYS	=	working days in a year. For consistency the same variable is used as in the ESAL calculation.

The extra annual VOC including the speed cycle and idling effects for the three groups at Year i is accomplished by multiplying a factor of 1.35:

$$VOC_i = 1.35 \times (VOCA_i + VOCB_i + VOCC_i) \qquad (6.55)$$

where:

VOC_i	=	extra annual VOC for all vehicles in Year i, \$/km;
$VOCA_i$	=	extra VOC in Year i for Group A, \$/km;
$VOCB_i$	=	extra VOC in Year i for Group B, \$/km, and
$VOCC_i$	=	extra VOC in Year i for Group C, \$/km.

The present worth of the extra VOC at each year is

$$PWVOC_i = VOC_i /(1+r)^i \qquad (6.56)$$

where:

$PWVOC_i$	=	present worth of extra VOC in Year i, \$/km;
VOC_i	=	extra VOC in Year i, \$/km;
r	=	discount rate, and
i	=	number of years from the present time to the VOC calculating year.

The present worth of the total extra VOC for all years is

$$PWVOC = \sum PWVOC_i \tag{6.57}$$

where:

PWVOC	=	present worth of the total VOC throughout the analysis period, \$/km,
$PWVOC_i$	=	present worth of VOC in Year i, \$/km.

6.3 Total Cost

The total cost refers to the sum of agency costs and user costs. As stated before, agency costs include the initial construction cost, rehabilitation cost, maintenance cost and the residual value as a negative cost, while road user costs include vehicle operating cost and user delay cost. Therefore, for each design alternative the present worth of total costs can be calculated as:

$$PWTTC = PWINC + PWRHC + PWMC - PWRSC + PWVOC + PWDLC \tag{6.58}$$

where:

PWTTC	=	present worth of total cost, \$/km;
PWINC	=	present worth of initial construction cost, \$/km;
PWRHC	=	present worth of total rehabilitation cost, \$/km;
PWMC	=	present worth of total maintenance cost, \$/km;
PWRSC	=	present worth of residual cost, \$/km;
PWVOC	=	present worth of total extra VOC, \$/km, and
PWDLC	=	present worth of total delay cost, \$/km.

OPAC 2000 uses the least total cost criterion to determine an optimal design alternative. Therefore in the output the available pavement design alternatives are ranked from the alternative with the least total cost to the one with the highest, while the user can select the number of design alternatives he/she would like to evaluate in the output. It should be noted that OPAC 2000 provides the option of including or excluding the vehicle operating cost and/or the user delay cost in the total cost calculation.

6.4 Vehicle Emission Model

A vehicle emission model for estimating the increased air pollution during pavement rehabilitation constructions on highways[18] is described in this section. The main pollution components considered in the model include carbon monoxide (CO) and hydrocarbons (HC). Other pollutants, such as nitrogen oxides (NO) and certain metallic compounds, also exist, but they are significantly less than the amount of CO and HC generated by the passing vehicle during the pavement rehabilitation.

To highlight pavement strategy comparison, a basic assumption applied in the emission model in OPAC 2000 is that within the analysis period emission peak hours happen in the overlay construction period. In other words, low vehicle speeds due to resurfacing activities are a major factor to be examined in developing the emission model. It is realized that the emission level of CO and HC pollutants is also related to many other factors which are beyond the scope of this research.

The basic relationship underlying the emission model is that pavement overlay activities cause vehicle speed to drop and then the lower vehicle speed causes a higher emission level. The emission model takes into consideration the selected traffic control plan and vehicle fleet speed prediction based on capacity. It gives the emitted amount the two pollutants as output. The model is based on the study in the Highway Performance Monitoring System (HPMS) [FHWA 1987] which links vehicle emission with the fleet speed. A step by step description is provided here to demonstrate how the emission model works:

Step 1: Choose a traffic control plan corresponding to the number of lanes on the divided highway:

[18] Only highways with more than 4 lanes are considered because two-lane highways, with lower traffic volumes, would have substantially less extra vehicle emissions due to rehabilitation.

4-Lane Highway	6-Lane Highway	8-Lane Highway
Traffic Plan 5	Traffic Plan 6	Traffic Plan 8

Step 2: Determine the normal capacity $CAPn_j$ and the reduced capacity $CAPr_j$ corresponding to the selected traffic control Plan j, as specified previously in the capacity model.

Step 3: Predict the AADT and hourly volume HV during the working time at the rehabilitation years.

$$HV = AADT_i \times 0.5 \times 1.2 \times 0.07$$

where:

$AADT_i$ is the annual average traffic at Year i

Step 4: Predict the normal speed Vn_j and the reduced speed Vr_j following the same procedure as described previously in the speed model.

Step 5: Given vehicle speeds, predict CO and HC emission amount (kg/1000 veh-km) by vehicle types using the HPMS emission model [FHWA 1987].

In the emission model vehicles are divided into the same three groups as in the VOC model. The CO and HC emission amount by Groups A, B and C in the direction without construction is noted as COAn, COBn, COCn and HCAn, HCBn and HCCn, respectively; and the CO and HC emission amount by the vehicle groups in the direction with construction is noted as COAr, COBr, COCr and HCAr HCBr, HCCr, respectively.

Step 6: Calculate CO and HC emissions (kg/km-h) of all vehicles from both directions. Equations (6.58) and (6.59) are used to calculate the two pollutants:

$$CO = [(COAn \times TYA + COBn \times TYB + COCn \times TYC) + (COAr \times TYA + COBr \times TYB + COCr \times TYC)] \times HV/1000 \quad (6.59)$$

$$HC = [(HCAn \times TYA + HCBn \times TYB + HCCn \times TYC) + (HCAr \times TYA + HCBr \times TYB + HCCr \times TYC)] \times HV/1000 \quad (6.60)$$

where:

TYA, TYB, and TYC = percentages of AADT for Groups A, B, and C, respectively.

Following the same procedure the CO and the HC emissions at the second or later rehabilitation period(s), if any, can be predicted so that peak emissions throughout the analysis period can be estimated for each pavement design alternative.

It may be noted, as for the VOC and user delay cost calculations, the computer package provides the option of including or excluding vehicle emission calculations.

6.5 Sample Analysis

The 6-lane highway in the previous flexible pavement design example is taken for the life-cycle cost analysis. The material unit costs for the four layers in the pavement structure are given in the following table:

Table 6.5 Material Unit Cost for the Sample Problem

Layer	Thickness (mm)	Density (Tonne/m^3)	Unit cost ($/Tonne)	Salvage value (%)
1	50	2.3	50	20
2	110	2.3	40	20
3	150	2.15	8	20
4	375	2.15	6	20
Future Overlay	75	2.3	50	20

When the unit cost is entered as $/Tonne, a density factor is used to convert it into the volumetric price. A twenty percent (20%) salvage value is used in the input for all the layers, which represents the reusable materials at the end of the analysis period. The routine maintenance cost is estimated as $1,000/lane-km at the base year, and increasing yearly at a 5% rate. In addition to this, a scheduled one-time maintenance cost of $10,000/lane-km is estimated for every fifth year after the initial and future overlay constructions. An annual discount rate of 6% is used for converting the costs into the present worth. The result is given in Table 6.6.

The two components in the residual value are the salvage value at Year 30 and the terminal value of the unused pavement life which is evaluated as a portion of the second overlay cost at Year 22. The total cost of the design alternative under evaluation is the summation of the present worth of the above cost elements, which is equal to $2,266,041/km.

Table 6.6 Life Cycle Cost Analysis Output for the Sample Problem

Year	Initial Cost	Rehab. Cost	Maint. Cost	Scheduled Maint. Cost	Residual Cost	Delay Cost	VOC
0	$627,469						
1			$ 6,000				$27,268
2			$ 6,300				$34,589
3			$ 6,615				$42,858
4			$ 6,946				$52,206
5			$ 7,293	$10,000			$62,789
6			$ 7,658				$74,807
7			$ 8,041				$88,515
8			$ 8,443				$104,249
9			$ 8,865				$122,461
10			$ 9,308	$10,000			$143,776
11			$ 9,773				$169,070
12			$ 10,262				$199,607
13		$194,063	$ 6,000			$934	$50,128
14			$ 6,300				$63,506
15			$ 6,615				$78,892
16			$ 6,946				$96,720
17			$ 7,293				$117,621
18			$ 7,658	$10,000			$142,553
19			$ 8,041				$172,337
20			$ 8,443				$209,428
21			$ 8,865				$257,211
22		$194,063	$ 6,000		$60,035	$1,329	$71,347
23			$ 6,300				$88,356
24			$ 6,615				$107,449
25			$ 6,946				$128,878
26			$ 7,293				$152,949
27			$ 7,658	$10,000			$180,033
28			$ 8,041				$210,590
29			$ 8,443				$245,206
30			$ 8,865		$182,419		$284,640
Present Worth	$627,469	$144,837	$103,323	$18,634	-$48,421	$806	$1,419,393

The amount of vehicle emissions during rehabilitation periods are given in the following table:

Table 6.7 Estimated Emission Output for the Sample Problem

Rehabilitation Period	CO Emission (kg/km-h)	HC Emission (kg/km-h)
1st Overlay	7.62	1.36
2nd Overlay	10.85	1.94

In the design report the pavement design alternatives are ranked according to the present worth of their total costs along with the structural analysis results. A final design decision can be made based on the predicted performance and the life-cycle costs, as well as the estimated emission, if it is required by an environmental assessment.

CHAPTER 7 THE SOFTWARE SYSTEM DESIGN

One of the major tasks in the development of OPAC 2000 was the comprehensive system design for development of the software package. The foundation of this package is the various structural and economic analysis modules as well as their required input data, as described in detail in the previous chapters. The focus in this chapter will be on the general system design and the report module.

The platform of the operating system for OPAC 2000 was selected to be Microsoft® Windows. Because system coding represented a large and extensive effort, a sub-contract to the University of Waterloo was arranged with ITX Stanley Ltd. (formerly Pavement Management Systems Ltd.) located in Cambridge, Ontario. The coding work was carried out by programmers in the firm, under the direction of the writer of this thesis.

7.1 General System Layout

As indicated earlier in Figure 3.1 the traffic, economic and other project data have to be entered into the system for the structural and economic analysis modules to function, and the analysis results become outputs to various design reports. The connections between the modules and user interfaces need to be specified. In addition, users should be able to access the system help message. All these tasks are organized through the system design. The following task groups are designed in the OPAC 2000 software package:

1. File: file utilities and system setup
2. Input Windows
3. Analysis Modules
4. Report
5. Help: on-line help

These task groups are designed in such a way that they can be translated into menus and submenus during the system coding. The use of the task groupings makes it easy to follow the MS

Windows™'s convention of designing the user interfaces. More details are given in this section except for the On-Line Help which is basically the contents of the User's Guide.

It may be noted that a comprehensive users guide was written as a part of the software package development. The Guide is contained in Appendix J, and provides detailed examples of all the elements of this chapter.

7.1.1 File and System Management

The "File and system management" group has two functions: (1) File management, which is similar to most of the Windows programs and is used to start a new project, to open an existing design document and to save the analysis report when the design is finished; (2) System management,

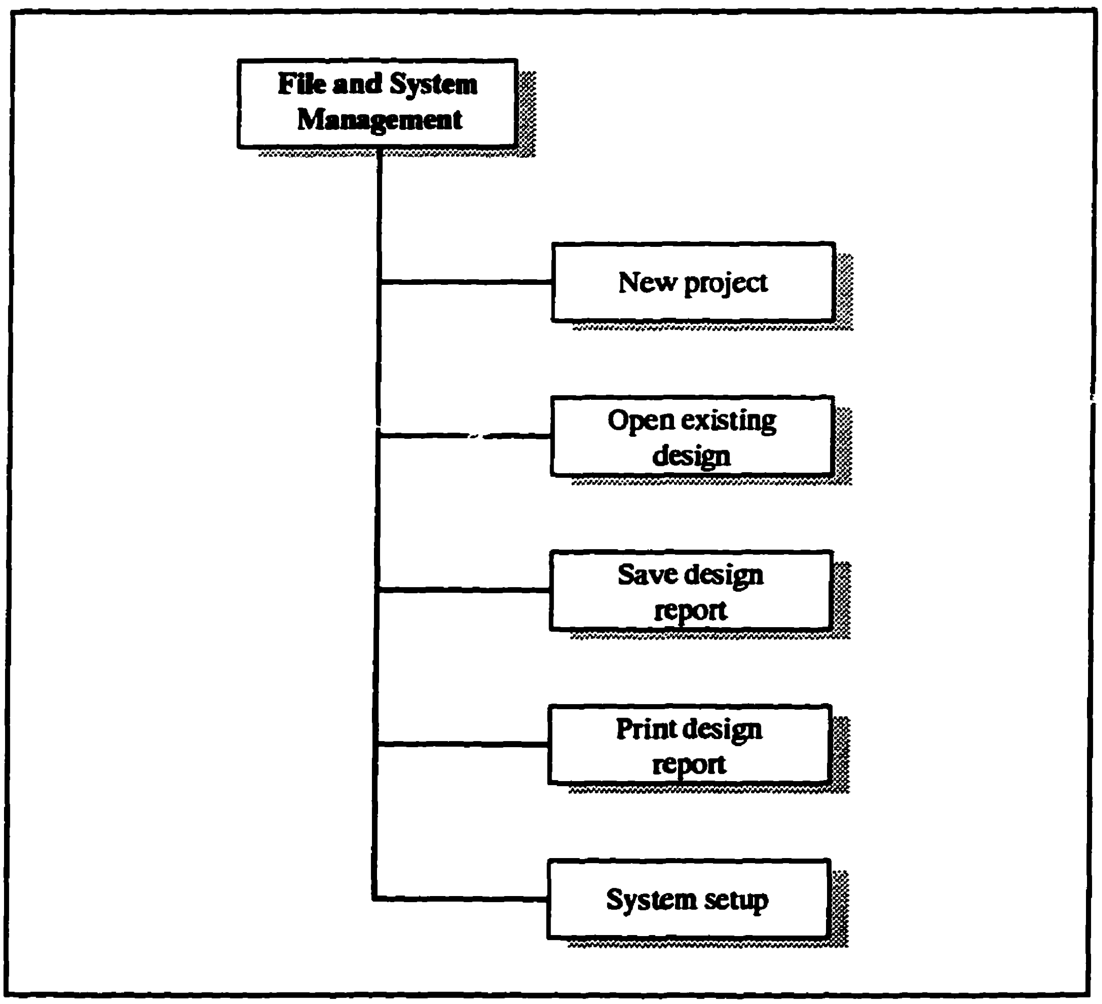


Figure 7. 1 The Task Group of File and System Management

such as printing the report and setting up the system defaults, etc. Figure 7.1 shows the organization of this group.

7.1.2. Input Windows

Input windows are designed to provide the user interface for three types of input information: (1) Project information, such as project name (highway number), section ID, number of lanes and lane width, as well as design criteria including performance levels, initial life requirements, the reliability level and the analysis period, etc.; (2) Traffic loading information, which allows the user to enter values of the variables identified in Chapter 4 to calculate the ESALs; (3) Economic analysis information, such as the available funds for initial construction, maintenance cost and discount rate, etc. Figure 7.2 shows the organization of this group.

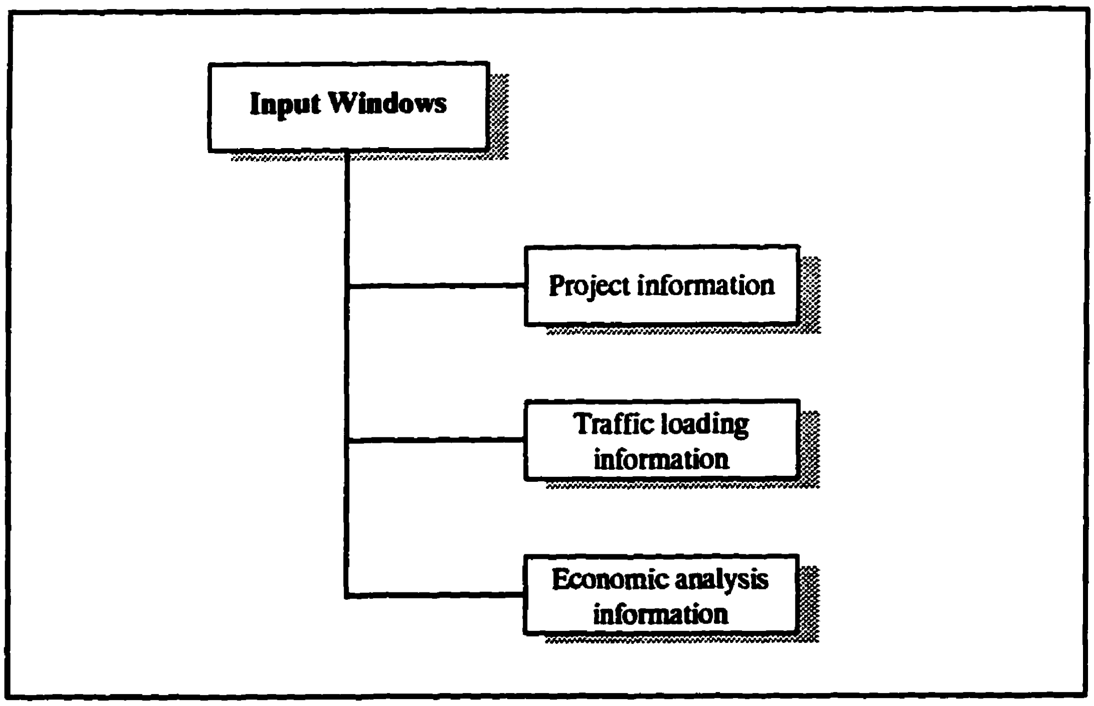


Figure 7. 2 The Task Group of Input Windows

7.1.3. Analysis Modules

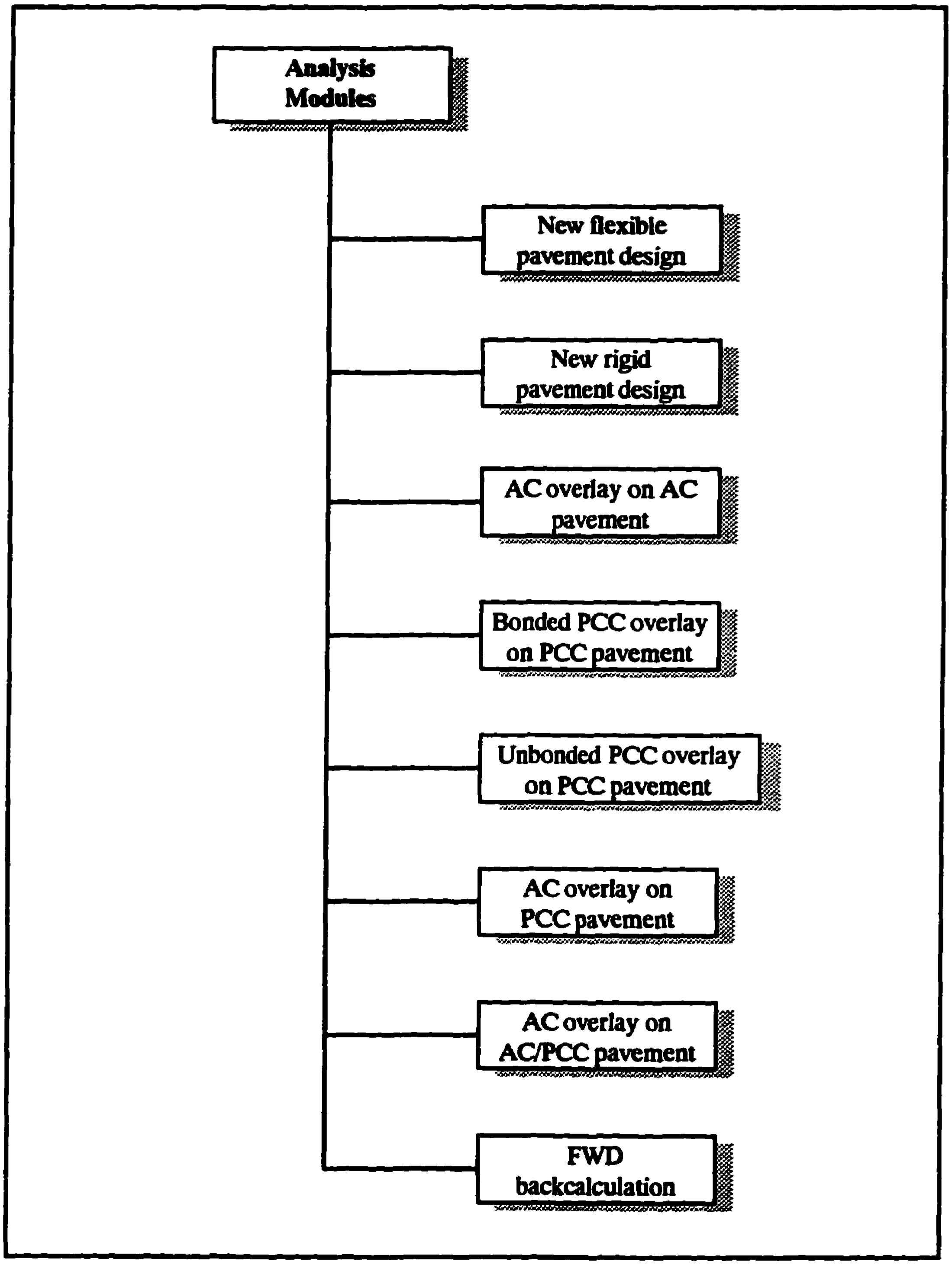


Figure 7. 3 The Task Group of Analysis Modules

The "Analysis" group provides access to 8 analysis modules. The first 7 are the pavement design modules including new flexible pavement, new rigid pavement, AC overlay on AC pavement, bonded PCC overlay on PCC pavement, unbonded PCC overlay on PCC pavement, AC overlay on PCC pavement and AC overlay on AC/PCC pavement. The range of layer thickness, material moduli and subgrade condition are entered with individual design modules.

The eighth module is the FWD analysis, for which the details are described in Chapter 5. Figure 7.3 shows the organization of this group.

7.1.4. Report

The "Report" task group has four functions: Input report, Output report, Sensitivity report and Cross-section report. While more details on the report are given separately in the next section, Figure 7.4 shows the organization of this group.

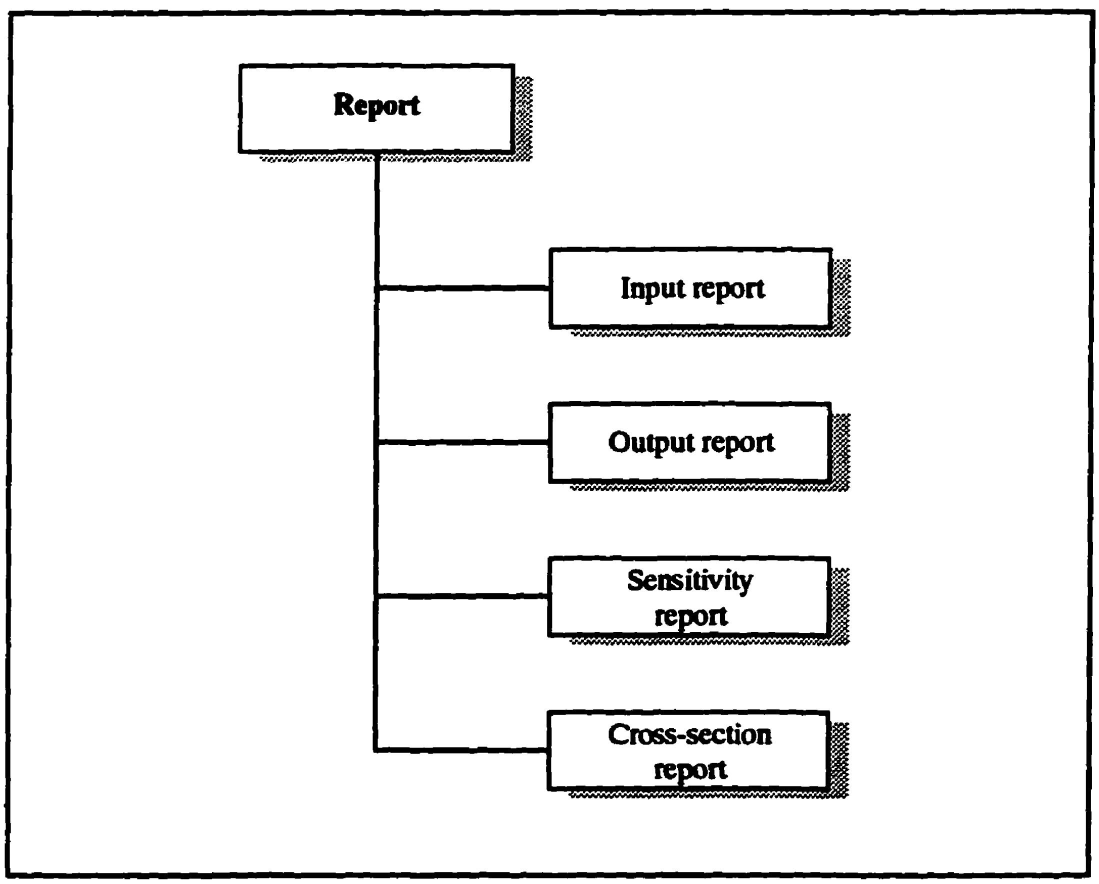


Figure 7. 4 The Task Group of Report

7.2 Report Module

A comprehensive report module is designed in OPAC 2000 to handle the data entered into the system as well as the data from the structural and economic analyses with the help of an internal database. The design report module is used to organize the data so that they can be presented in different categories, as follows:

1. Input Report
2. Design Alternatives Report (Output Report)
3. Sensitivity Analysis Report
4. Graphics Outputs

The following discussion provides a brief summary of these reports which helps to answer the question "What the system is able to provide as the output". Actual example screens for these reports are given in the OPAC 2000 User's Guide in Appendix J.

7.2.1 Input Report

The input report is used to summarize all the information that the pavement designer put into the system in order to perform the design analyses. The input data is arranged in sections such as:

1. Project Description: the project location, section ID, the cross-sectional information, the performance criteria and reliability, the designer and the design date,
2. Traffic Information: initial AADT, growth rate and the breakdown of the truck traffic,
3. Design and Economic Information: materials of layers, layer thickness limits and increments, unit costs, maintenance schedule and costs, information on future rehabilitation(s) and shoulders, etc.

The input report can be viewed on the computer screen and printed as a hard copy, so that the design engineer can have a quick review of the input information and determine if any adjustment is needed for further analysis. It can also be used as a part of the pavement design document.

7.2.2 Design Alternatives Report

The design alternatives report summarizes the structural and economic analysis results from running the system. The following information is shown in the report for all the seven types of pavement designs in OPAC 2000:

1. Project Description: the type of design, project ID, the designer and the design date,
2. Project Costs: agency costs (initial construction cost, maintenance cost, rehabilitation cost), user costs (user delay cost and VOC), residual value (as a negative cost) and total cost,
3. Pavement Structure: layer thicknesses of each design alternative, in terms of both the actual layer depth and the equivalent thickness,
4. Performance Time: life-span of the initial pavement structure as well as after each future overlay, etc.

All the design results are arranged in columns corresponding to each pavement design alternative, and the design alternatives are ranked in an ascending order according to their total costs. Similar to the input report, the design alternatives report can be viewed both on the computer screen and be printed as a hard copy. It can also be used as a part of the pavement design document.

7.2.3 Sensitivity Analysis Report

The OPAC 2000 sensitivity analysis module provides a tool to examine the impact of a design variable on the design output. Four independent design variables are considered in the sensitivity analysis: the minimum acceptable PCI, design reliability, initial AADT and traffic growth rate. The analysis outputs to be examined are the total cost, the agency cost and the user cost.

Sensitivity analysis is performed after the structural and economic analyses. The designer needs to select a pavement design alternative from the output report and a design variable from the four independent variables mentioned above. The program will select a range of the variable and recalculate the total cost, the agency cost and the user cost for the given pavement design alternative. The result of sensitivity analysis is plotted over the range of the selected independent variables. Sensitivity analysis can be performed for all the design alternatives at the user's choice. Examples on sensitivity analysis can be found in the OPAC 2000 User's Guide.

7.2.4 Graphics Outputs

Graphic outputs are available for the predicted pavement performance, the cross section and the sensitivity analysis. They are accessed through the drop-down menus of the system. To activate a graphic output the design analyses need to be performed first, and the user specifies a design alternative number so that the pavement performance curve, the cross-section or the sensitivity analysis result can be plotted. Some sample graphics can be found in the OPAC 2000 User's Guide.

7.3 Software Package

The actual software package is effectively described in Appendix J, as previously noted. It was initially tested in-house (alpha testing), and then with a selected group of regional and head office MTO users (beta testing). As well, spreadsheet calculations for all elements of the package, using the engineering models and economic analysis models described in previous chapters of the thesis, were carried out to verify the outputs of the package. The result is Version 1.0 of the software package, which is intended to be installed in all MTO regions in the near future.

CHAPTER 8 EXTENDING OPAC 2000 TO NETWORK LEVEL PAVEMENT MANAGEMENT

The objective of network level pavement management is to answer two types of questions: (1) the "what if" question: to investigate the relationship between various funding levels and the pavement network status, and (2) the "what to do" question: while a pavement management system itself can not make decisions, it can provide a network level pavement work program based on a set of criteria to support the decision-making.

As shown previously in Figure 2.1 project level pavement design is performed for those pavement sections selected "on-line" from the network level analysis. In an ideal pavement management system the project level analysis on the selected needs sections (sections which need rehabilitation), including the corresponding rehabilitation strategies and costs from the network level analysis, should not result in major modifications. In reality, however, discrepancies often exist between the decisions made at the two levels. A basic reason is that different models are used at the two levels, particularly those regarding performance predictions. Network analysis is not practical with "data hungry" models; that is, where a great deal of time, effort and costs are required to obtain the input data. The flexible pavement performance prediction model in OPAC 2000 does offer the potential, however, to be used at the network level.

The purpose of this chapter is to demonstrate how the OPAC 2000 pavement performance prediction model is also applicable in the network level pavement management, and thereby provide a way to reduce the discrepancy between analysis carried out in the two levels.

8.1 Sample Network and Assumptions

To limit the amount of calculations, a subset of the 17 sections from northern Ontario is selected to form a sample network. The detailed information in terms of the pavement structure, the subgrade and the traffic is given in Appendices B and C. The sections are re-numbered as listed in Table 8.1, to facilitate the ensuing analyses.

Network level analyses include Needs Analysis, Rehabilitation Analysis and Optimization Analysis. Needs analysis identifies the pavement sections in the network which have reached a minimum acceptable PCI level and are therefore "due" for rehabilitation. Rehabilitation analysis determines the costs and benefit of different rehabilitation alternatives for each pavement section. Needs and rehabilitation analyses provide the input for the optimization analysis which is used to answer the two types of questions mentioned at the beginning of this chapter.

Table 8.1 Sample Sections for Network Level Analysis

No.	Section ID	Length	AADT	No.	Section ID	Length	AADT
N1	NR_11_17030(N)	6.0	4900	N10	NR_14_40200()	35.6	500
N2	NR_11_17030(S)	6.0	4475	N11	NR_11_42500()	9.2	800
N3	NR_11_17060(N)0.00	10.3	5800	N12	NR_14_46480()	15.9	2300
N4	NR_11_17060(N)10.30	11.6	5350	N13	NR_14_67800()	10.1	9500
N5	NR_11_17060(S)10.30	11.6	5600	N14	NW_19_18170()10.40	11.3	900
N6	NR_11_28010()	16.0	1300	N15	NW_19_18170()21.70	14.9	525
N7	NR_11_33330()0.00	9.8	1300	N16	NW_18_21360()	17.6	2300
N8	NR_11_33330()9.80	11.0	1300	N17	NW_18_21554()	27.5	1650
N9	NR_11_35380()	19.1	6750				

Pavement performance data of several sections in the selected subset are dated back in the 1970's to 1980's. To make the group of pavement sections within a relatively closer time frame, they are shifted 10 years forward. The sections being shifted include: N1, N2, N4, N5, N6, N11, N12 and N16. Furthermore, the following assumptions are made for the network analysis:

1. The minimum acceptable performance level is selected at PCI = 60 for the needs analysis and the rehabilitation analysis,
2. Three rehabilitation treatment alternatives are considered for each pavement section in the study:
   - Thin asphalt hot mix overlay of 25 mm,
   - Medium asphalt hot mix overlay of 75 mm,
   - Thick asphalt hot mix overlay of 125 mm,

   The material unit cost (HL1) is $40 per cubic meter.

3. The structural analysis period is 10 years, or in other words, pavement performance is analyzed for all the sections during the period of 1991 to 2000,
4. Budget programming period is 5 years from 1991 to 1995. Two levels of expected annual rehabilitation budget, \$500,000 and \$1,000,000 are considered in the study. They are assumed to remain constant during the 5-year period,
5. If there is insufficient budget in a given year, the maximum delay of a rehabilitation treatment is 3 years from the needs year.

8.2 Needs Year and Rehabilitation Analysis

Needs year refers to the year in which a pavement section deteriorates to the minimum acceptable performance level, which in this case, is PCI = 60. The first step in the network level analysis is to find the portion of the network which is or will be needs in the analysis period. This is accomplished by pavement performance prediction. Since all the sections in the sample network are located in northern Ontario, the performance prediction model in Equations (4.1), (4.4) and (4.7) with coefficients $\beta = 10.5478$ and $\alpha = -0.0415$ from Table 4.3 are used in the pavement performance predictions. Table 8.2 contains the result of the performance predictions.

Table 8.2 Sections and Their Needs Years

Year	Sections	Length (2 lane-km)	Percent of the network
1991	N8, N9	30.1	12.4
1992	N3	10.3	4.2
1993	N13, N14, N15	36.3	14.9
1994	N2, N11	15.2	6.2
1995	N6, N17	43.5	17.9
1996	N1	6.0	2.5
1997	N/A	0	0
1998	N7, N16	27.4	11.3
1999	N4	11.6	4.8
2000	N5, N12	27.5	11.3
2001	N10	35.6	14.6
Sum	17	243.5	100

Figure 8.1 shows the percentage distribution of the network needs, in which 56% become needs sections in the first 5 years and the rest become needs for rehabilitation in the later years.

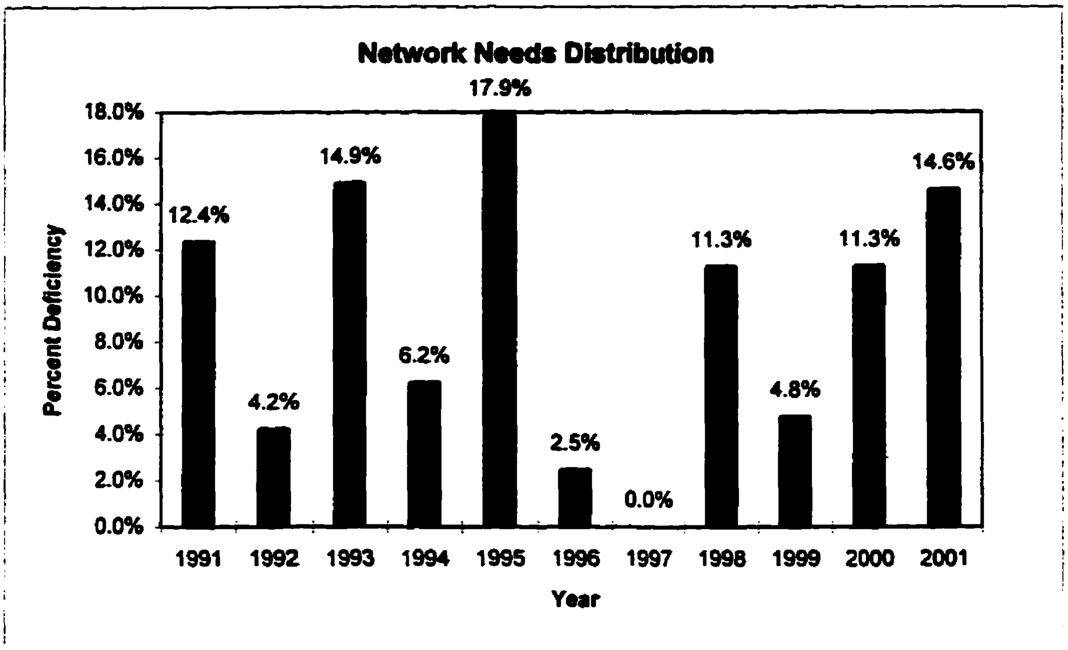


Figure 8. 1 Network Needs Distribution

The step following the needs analysis is the rehabilitation analysis which serves the purpose of determining the benefits and costs of different rehabilitation alternatives for each pavement section. In general the benefit of pavement rehabilitation can be represented by the reduced cost to road users in terms of the vehicle operating cost (VOC). Since VOC is closely related to the predicted pavement performance, a simple and convenient surrogate measure that is commonly used in practice is the area under the pavement performance curve, weighted by traffic volume and section length, as shown in Figure 8.2 [Haas 1994]. The area is termed as the "effectiveness" of a rehabilitation alternative. Basically the higher the performance curve, the longer is the life span of a rehabilitation alternative, which results in higher effectiveness or greater benefits to the road user.

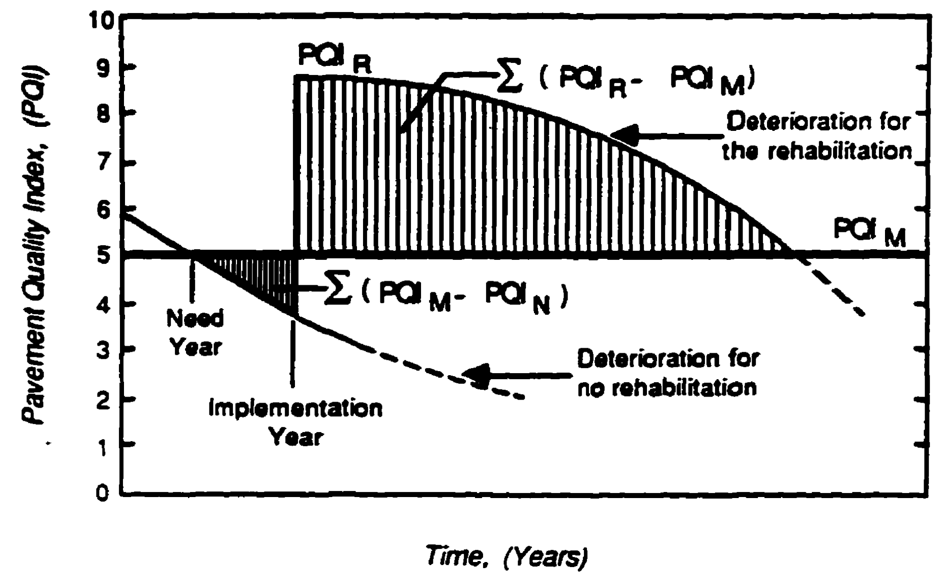


Figure 8. 2 Effectiveness of Pavement Rehabilitation

As happens commonly in practice, a pavement rehabilitation may not be implemented at the needs year due to budget constraints. In this case the pavement will continue to deteriorate until the implementation year. The area below the minimum acceptable line in Figure 8.2 represents negative effectiveness or the negative benefit to the users. The total effectiveness is calculated using the following equation [Haas 1994]:

$$Effectiveness = \left\{ \sum_{RehabYr}^{PCI_R \geq PCI_M} (PCI_R - PCI_M) - \left[\sum_{PCI_N \geq PCI_M}^{RehabYr} (PCI_M - PCI_N) \right] \right\}$$

$$\times \{AADT\} \times \{ Length\ of Section \} \qquad (8.1)$$

where:

PCI_R = Pavement Condition Index (PCI) after rehabilitation (i.e., for the implementation year) and for each year until PCI_M is reached,

PCI_M = minimum acceptable level of PCI,

PCI_N = yearly PCI from the needs year to the implementation year.

The effectiveness calculated using Equation 8.1 in actuality has mixed units, but it is used as a unitless quantity. Because a maximum delay of 3 years is considered in this study, the effectiveness is calculated for the needs year and for the ensuing three possible implementation years for each rehabilitation alternative and each pavement section based on the predicted performance from OPAC 2000.

The project cost calculation is carried out by multiplying the length of the section with the unit cost output (one kilometer in length) of the OPAC 2000 package. The maintenance cost is not considered since it is not the main theme of the chapter. User costs are not considered due to the fact that the effectiveness calculations are used to account for the rehabilitation benefits. The effectiveness and project cost calculation results can be found in Appendix H (note: the cut-off year is 2001 for the effectiveness calculation).

8.3 Optimization Analysis with Integer Programming

The optimization analysis uses the results from the needs and rehabilitation analyses. While the main objective of optimization analysis is to provide a network level strategy that maximizes the benefits, it can be further divided into the following two functions:

(1) Investigate the effect of various funding levels on the network performance

(2) Develop pavement working programs corresponding to the funding levels

An Integer Programming (IP) method is used for the network optimization analysis in this study. The IP formulation, as introduced in this section, is followed by the interpretation of the results with respect to the above mentioned two functions.

An IP model with 0-1 decision variables is applicable to this study because a pavement rehabilitation alternative can either be selected or not selected. The decision variables in the problem are defined as:

N(*ijt*): 0 - 1 decision variable, "1" means the rehabilitation alternative is selected by the optimization procedure, and "0" means the rehabilitation alternative is not selected,

i = 1...17, one of the pavement sections in the network,

j = A, B or C, pavement rehabilitation alternatives: thin overlay, medium overlay and thick overlay, respectively,

t = 1991...1995, the year rehabilitation alternative *j* is selected for pavement section *i*.

For the five-year programming period the IP model can be expressed as (based on [Haas 1994]):

$$\text{Maximize} \quad \sum_{i=1}^{17}\sum_{j=A}^{C}\sum_{t=1991}^{1995} N_{ijt}\cdot E_{ijt} \tag{8.2}$$

$$\text{Subject to:} \quad \sum_{j=A}^{C}\sum_{t=1991}^{1995} N_{ijt} \leq 1 \qquad \text{for } i = 1\ldots17 \tag{8.3}$$

$$\sum_{i=1}^{17}\sum_{j=A}^{C} N_{ijt}\cdot C_{ijt} \leq B_t \qquad \text{for } t = 1991\ldots1995 \tag{8.4}$$

$$\text{and} \quad N_{ijt} \geq 0 \tag{8.5}$$

where:

N_{ijt} = Section *i* with rehabilitation alternative *j*, in year *t*

E_{ijt} = Effectiveness of section *i*, with rehabilitation alternative *j*, in year *t*

C_{ijt} = Cost of section *i*, with rehabilitation alternative *j*, in year *t*

B_t = budget for year *t*.

In the above IP model, Equation (8.2) is the objective function for maximizing the effectiveness. Equation (8.3) is the uniqueness constraint which requires that at maximum one rehabilitation alternative can be selected once in the programming period for any pavement section. Equation (8.4) states that the cost of all the rehabilitation alternatives selected in a given year can not exceed the budget limit of that year. Equation (8.5) is the non-negative constraint which is required by all Integer Programming models.

The solution of the IP model is obtained through the LINDO computer software. The analysis is performed at two budget levels: yearly budget level $500,000 and $1,000,000, and the output of the LINDO is included in Appendix I.

To answer the "what if" question, Figure 8.3 shows that with the first budget level the average network performance will be raised to slightly above PCI = 75 at the end of 5 years. With budget level 2, the network performance will be raised even higher to about PCI = 80 at the end of the 5 year programming period. For comparison purposes the network condition without rehabilitation (or no build) is also predicted in Figure 8.3. It shows the performance would be lowered substantially to about PCI = 63 by the year 1995.

Another way to look at the function of network optimization analysis is for the IP model to minimize the network rehabilitation cost subject to constraints that the network performance will be better than or equal to a specified performance standard. This approach is useful to find out the budget requirements, but it was not performed in this study.

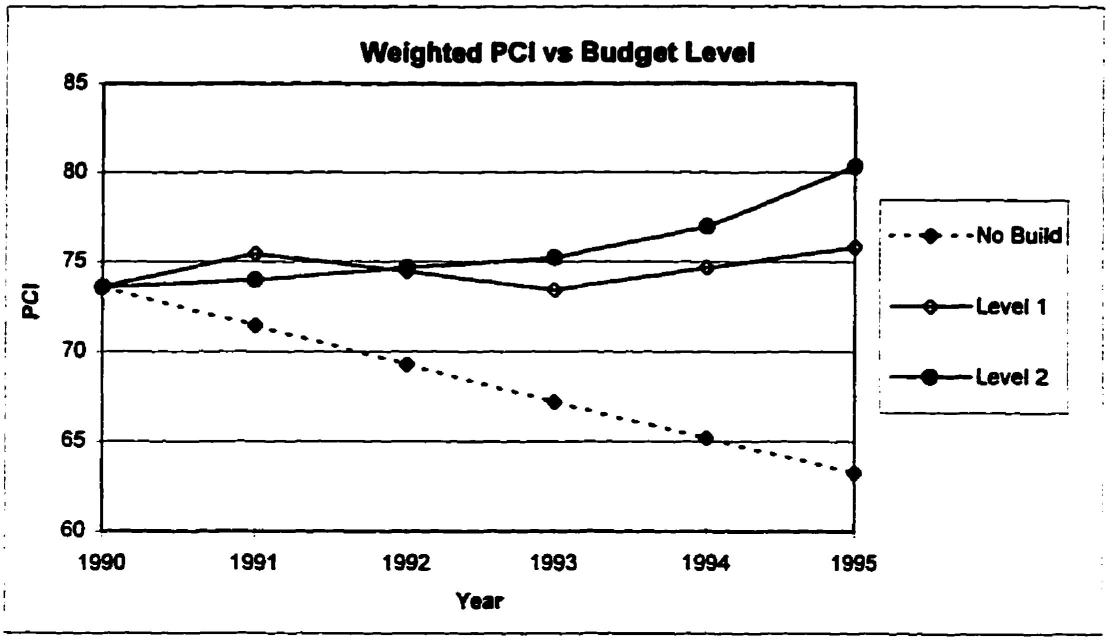


Figure 8.3 Network Performance Vs. Budget Level

The "what to do" question involves providing a network working program in responding to "3 W's": *what* section to improve, *what* alternative to select and at *what* time to implement. In the process of solving the IP model the potential rehabilitation projects are evaluated so that the sections, within-project alternatives and their timing are determined to maximize the overall network effectiveness or benefits subject to the given budget levels. Table 8.3 provides the priority list on the two budget levels based on the solution to the above IP model. An action plan can then be developed based on the selected budget level and the corresponding priority list.

Table 8.3 Network Rehabilitation Program Priority List

Year	Budget Level 1: $500,000/year			Budget Level 2: $1,000,000/year		
	Section	Alternative	Cost ($)	Section	Alternative	Cost ($)
1991	N8 N9	A A	484,610	N9	B	922,530
1992	N3	B	497,490	N8 N3	A B	967,459
1993	N13	B	487,830	N13 N14	C A	994,980
1994	N14 N2 N11	A A A	426,650	N15 N2 N11	A C A	871,010
1995	N17	A	442,750	N6 N17	A A	700,350
Sum	8		2,339,330	10		4,456,329

8.4 Summary

As a summary to Chapter 8, the network level pavement management involves needs, rehabilitation and optimization analyses. The OPAC 2000 pavement performance prediction model developed for the project level pavement designs can also be used in the network level needs and rehabilitation analyses. Since project level analysis is a continuation of the network level analysis, a major benefit of applying the OPAC 2000 pavement performance prediction model in the network level analysis is to help reduce the difference between the decisions made at the two levels.

CHAPTER 9 CONCLUSIONS AND RECOMMENDATIONS

Development of the OPAC 2000 pavement design system is presented in this thesis. This comprehensive new system is the first major update to Ontario's existing OPAC pavement design system. The key enhancements include:

1. A new set of OPAC 2000 flexible pavement performance models,
2. Capability of carrying out overlay designs,
3. Capability of carrying out reliability analysis,
4. Capability of carrying out rigid pavement design including overlay designs by employing the AASHTO rigid pavement design equation,
5. A new, improved economic analysis module,
6. Capacity of estimating impacts to environment due to pavement works, and
7. Use of the MS Windows™-based computing environment, and
8. A versatile, "user-friendly" software package.

This thesis provides the procedures, equations and the related background engineering principles that were used in the development of the system. The following conclusions are made based on the study:

1. The mechanistic-empirical nature of the OPAC pavement design method is retained in the OPAC 2000 pavement design system,
2. The OPAC pavement performance prediction model is updated based on in-service pavement performance data. Two separate models are developed based on cluster analysis: one for Southern Ontario, and one for Northern Ontario,
3. A systematic methodology was used in developing OPAC 2000 as a fully functional self-contained pavement design package,
4. A project level pavement design system should be considered within the scope of an overall pavement management system.

Although efforts have been made to make full use of the available data and the state-of-art technology in designing the system, the following areas are recommended to be considered for further enhancements in the future.

Data Base

Data collection represented a major and time consuming part in the study. For the structural data, some of the layer thickness and subgrade data are missing on the MTO pavement performance fact sheets, and visits to the regional pavement engineers had to be made for more complete information. Because of their importance in performance model building or updating, enough time should be planned to collect the data in future updating projects, perhaps with the help of coring tests (coring tests are offered by the Road and Development Branch in MTO, but were not made due to the time limits). For the traffic data, AADT and commercial vehicle percentage (trucks) data were provided by the Traffic Management Programming Office. Advice was given, however, that the commercial vehicle data are not of high quality. With the Ministry switching to the FHWA vehicle classifying system, it is hoped that better traffic data will be available for future model calibrations.

Traffic Associated Performance Loss Model

The traffic related part of the flexible pavement performance prediction model remained unchanged in the model calibration. The main reason is that it was based on curve fittings to the AASHTO Road Test data [Jung 1974] which was basically a load-intensive test. The AASHO Road Test is considered still to be the best source of this type. It is recommended, however, that this part of the model to be calibrated in future updates when better data sources are made available (for example, the SHRP data or data from other specialized projects), and the environmental part of the model should also be re-calibrated accordingly.

Material Properties and Drainage

Granular Base Equivalency (GBE) factors play an equally important role as the layer thickness data in the OPAC 2000 models. If handled properly, they should be able to characterize the properties of various types of paving materials. The research on this issue should be continued to keep up with the evolution of new materials and new construction methods, such as the cold in-place and hot in-place recycling, etc. A similar situation exists with the subgrade modulus, M_r. Rock-cut and rock-fill type of subgrades exist in the North part of Ontario, but the corresponding modulus (M_r) values are not available in the current MTO Pavement Design and rehabilitation Manual [MTO 1990]. OPAC 2000 has the provision of incorporating drainage coefficients for non-asphalt layers. Research on drainage coefficients should be carried out to provide appropriate inputs.

Performance Modeling As An On-Going Effort

Due to the changes in traffic loads and environmental factors, as well as the variability of pavement material properties, pavement performance modeling needs to be carried out in a progressive and iterative way. It is important to note that the OPAC 2000 model should go through several cycles of future calibrations based on long-term pavement performance data in order to further reduce the prediction error. This in turn emphasizes the importance of a good data base.

One related issue is to consider incorporating the OPAC 2000 pavement performance prediction model in the network level of pavement management. As pointed out in Chapter 8, the benefit will be to reduce the differences between the decisions made at the network level and the project level.

Finally, OPAC 2000 was developed specifically for the province of Ontario. Notwithstanding, the engineering principles, the techniques and the methodology used in developing the system are transferable to other regions. OPAC 2000 type systems could be used in such other regions through appropriate calibrations of the performance and economic analysis models.

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Appendix A

Sample of MTO Action Plan Fact Sheet and Pavement Performance Record

ACTION PLAN FACT SHEET

DISTRICT: 11
HIGHWAY : 11

LHRS	OFFSET	LENGTH	DIRECTION	FACILITY	CLASS	LANES	AADT	TRUCK %
17060	.00	10.3	N	A	E	2	11600	16.5

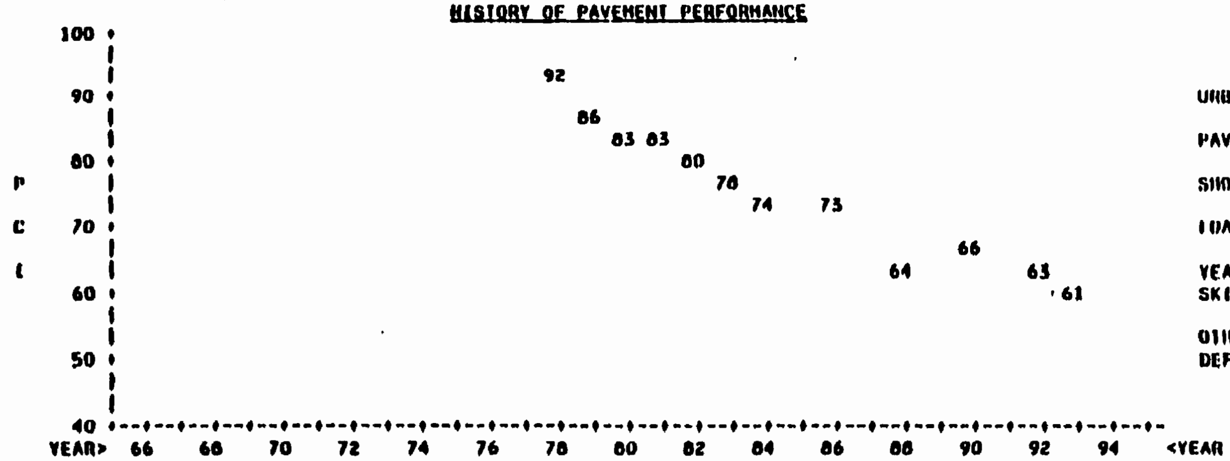
FROM: JCT. HWY 117
TO : SOUTH MARY LAKE ROAD

	STRUCTURE	REHABILITATIONS 1	2	3	4	5
CONTRACT NO.	75058					
SURFACE TYPE (CODE)	H2					
TOTAL SURFACE THICKNESS, MM	140					
ADDED OVERLAY THICKNESS, MM						
MILL / REMOVE THICKNESS, MM						
BASE TYPE (CODE)	GD					
BASE THICKNESS, MM	150					

SUBBASE TYPE (CODE) : GS
SUBBASE THICKNESS, MM : 450
SUBGRADE TYPE (CODE) : L

STRUCTURAL COMMENTS:

HISTORY OF PAVEMENT PERFORMANCE



URBAN SECTION, % : 0
PAVEMENT WIDTH,M : 7.3
SHOULDER TYPE : G
LOAD RESTRICTION :
YEAR OF LAST SKID TEST :
OTHER DEFICIENCIES :

PAVEMENT CONDITION:	YEAR	PCI	RCR	DMI
	1993	61	5.1	41.3

ESTIMATED CHANGE IN PCI AFTER 3 YEARS > 10 <

PREFERRED, HOLDING AND DEFERRED STRATEGIES

	PART	DESCRIPTION	CODE	PROG YEAR	EXTENT %	COST/KM ($1000)	PCI AFTER	LIFE YEARS	S
PREFERRED STRATEGY	I	FH H.I.P. 25MM OVERLAY		1994	100	75.0	95	15	
	II								
HOLDING STRATEGY	I	SELECTIVE RESURF & FH TREATMENT		1993	15	7.5		15	
	II	H.I.P. 25 MM OVERLAY		1996	100	75.0	95		
DEFERRED STRATEGY	I	FH MILL 40MM - 90MM RESURF		1996	100	135.0	95	15	
	II								

STRATEGY COMMENTS:

PREPARED BY:
DATE: MAY 12, 1994

PAVEMENT PERFORMANCE RECORD

DISTRICT: 11
HIGHWAY: 11

LHRS	OFFSET	LENGTH	DIRECTION	FACILITY	CLASS	LANES	AADT	TRUCK %
17060	.00	10.3	N	A	E	2	11600	16.5

FROM: JCT. HWY 117

TO : SOUTH MARY LAKE ROAD

OVERALL PAVEMENT PERFORMANCE HISTORY

YEAR	78	79	80	81	82	83	84	85	86	87	88	89	90	91	92	93	94	95
AGE	3	4	5	6	7	8	9		11		13	14	15	16	17	18		
PCR	85	85	85	85	85	85	85		80		70	76	72	65	69	60		
PCI	92	86	83	83	80	70	74		73		64		66		63	61		
RCR	9.0	8.0	8.0	8.0	8.0	8.0	6.8		6.4		5.6	7.5	5.0	6.5	5.4	5.1		
DMI	10.0	14.5	20.0	21.0	28.0	34.3	34.3		30.0		42.0	39.8	40.3	39.0	40.5	41.3		

DETAILED PAVEMENT PERFORMANCE HISTORY

SEV.=SEVERITY, DEN.=DENSITY		1988 SEV.	1988 DEN.	1989 SEV.	1989 DEN.	1990 SEV.	1990 DEN.	1991 SEV.	1991 DEN.	1992 SEV.	1992 DEN.	1993 SEV.	1993 DEN.	19 SEV.	19 DEN.
C. AGG. LOSS & RAV.		2	2	2	2	2	2	2	2	2	2	2	2		
FLUSHING															
RIPPLING AND SHOVING															
WHEEL TRACK RUTTING		3	4	3	3	3	3	2	4	2	3	2	3		
DISTORTION		3	2	3	2	3	3			2	1	2	1		
LONGITUDINAL	SING. & MULT.	3	2	3	2	2	2	2	3	2	3	2	3		
(WHEEL TRACK)	ALLIGATOR														
CENTRE LINE	SING. & MULT.	3	4	3	3	2	1	3	3	3	4	3	4		
	ALLIGATOR							2	1	2	1	2	1		
PAVEMENT EDGE	SING. & MULT.			2	1	3	3	2	2	2	2	2	2		
	ALLIGATOR														
TRANSVERSE	HALF,FULL & MULT.	2	4	3	3	3	3	3	3	3	3	3	3		
	ALLIGATOR							2	1	2	1	2	1		
MEANDER AND MIDLANE		3	1	3	2	2	1	2	2	2	2	3	2		
RANDOM								2	3	2	2	2	1		

SEVERITY OF DISTRESS CODES

1. VERY SLIGHT
2. SLIGHT
3. MODERATE
4. SEVERE
5. VERY SEVERE

DENSITY (EXTENT) CODES

1. <10%
2. 10-20
3. 20-50
4. 50-80
5. 80-100

MAJOR MAINTENANCE HISTORY

YEAR>	1993		
ITEM	I	II	III
TYPE(CODE)			
EXTENT %	0	0	0
COST	.0	.0	.0

SHOULDER CONDITION FOR GRAVEL SHOULDERS

YEAR> 1993
SEV. DEN.

DISTRESS COMMENTS: FEW SEVERE COARSE AGGREGATE LOSS.

EXTENT OF MAINTENANCE TREATMENT IN 1993 (CODE)

	PAVEMENT	SHOULDER
MANUAL PATCHING	0	0
MACHINE PATCHING	0	0
SPRAY PATCHING	0	
ROUT & SEAL CRACKS	0	0
CHIP SEAL	0	0

OTHER COMMENTS :

DATE: MAY 12, 1994

Appendix B

Pavement Section List

Region1:	SOUTHWEST REGION SECTIONS								
LHRS	OFFSET	Begin Yr	End Yr	delta PCI	Overlay	AC	GB	SB	sbgd
12900()	1.90	1973	1990	16.8	60	75	150	225	2
13595(N)	0.00	1973	1993	24.6		140	230	230	2
13595(S)	0.00	1973	1993	32.5		140	230	230	2
13605(N)	0.00	1974	1993	14.6		140	150	300	2
13605(S)	0.00	1974	1993	13.0		140	150	300	2
13620()	0.00	1971	1990	25.6		40	150	450	2
13630()	2.10	1979	1993	27.1	60	105	150	450	2
14510()	6.70	1970	1993	35.6		140	150	650	2
14530()	0.00	1977	1993	32.5		120	150	300	2
14570()	1.00	1973	1992	27.5	65	100	225	225	2
16220()	0.00	1975	1988	19.9		60	150	300	3
16220()	15.60	1975	1992	15.8		70	150	750	3
16970()	0.00	1974	1993	34.1		130	150	550	2
17000(N)	0.00	1979	1993	18.4		130	150	550	2
24110()	0.60	1976	1993	31.7	60	125	150	150	4
24570()	2.00	1974	1992	27.5	60	50	150	750	3
24600()	2.50	1982	1993	36.7	50	65	150	250	4
24960()	4.50	1973	1992	34.5	65	110	150	300	1
25480()	6.10	1981	1993	27.4		105	150	450	3
25620()	8.80	1974	1992	25.4		125	150	400	2
25950()	0.00	1972	1992	18.7		240	150	250	2
33010()	2.60	1973	1992	27.3	60	50	150	450	5
36600()	0.00	1974	1991	30.0	60	80	150	200	5
37300()	0.00	1982	1989	42.5	40	70	150	300	3
37320()	0.00	1985	1992	28.8	45	215	150	100	3
37620()	0.00	1975	1991	26.4	50	60	150	450	5
47740(E)	0.00	1976	1993	22.9	40	185	225	600	1
47740(W)	0.00	1976	1993	25.3	40	185	225	600	1

Region2:	CENTRAL REGION SECTIONS								
LHRS	OFFSET	Begin Yr	End Yr	delta PCI	Overlay	AC	GB	SB	sbgd
10014(W)	0.00	1984	1994	34.4	38	213	150	325	5
10840()	2.10	1973	1993	15.2		140	150	350	2
10840()	10.30	1971	1993	27.5		140	150	375	2
10850()	4.20	1982	1993	25.9		140	150	375	2
13170()	0.00	1983	1994	24.5	38	156	150	300	5
13180()	0.00	1982	1992	32.1	38	225	150	500	2
14330()	0.00	1971	1993	18.9		95	150	375	2
14330()	2.90	1971	1993	19.9		110	150	550	2
14421()	0.00	1973	1993	27.2		140	150	450	2
14460()	0.00	1979	1993	39.2	65	140	150	450	2
16120()	0.00	1977	1993	34.9	38	57	150	375	3
16470()	0.00	1984	1993	14.9	100	110	200	400	2
19380()	0.00	1978	1993	34.1	95	38	150	380	3
31030()	0.00	1974	1993	21.8	95	38	150	380	3
48500(S)	0.00	1979	1993	14.2		230	225	375	2
48500(N)	0.00	1979	1993	17.5		230	225	375	2
48515(S)	0.00	1981	1993	13.5		225	165	375	2

Region3:	EASTERN REGION SECTIONS								
LHRS	OFFSET	Begin Yr	End Yr	delta PCI	Overlay	AC	GB	SB	sbgd
10320()	6.10	1977	1993	40.9	60	120	150	300	2
10330()	3.20	1977	1993	50.9	60	120	150	300	2
10340()	0.00	1979	1993	47.5	65	95	150	300	2
10360()	0.00	1977	1991	40.1	60	95	150	150	2
14900()	0.00	1980	1992	42.5	100	80	150	375	2
14900()	9.00	1980	1992	32.0	100	100	150	300	2
20110()	1.20	1980	1989	39.8	40	110	150	150	2
20150()	0.00	1974	1993	35.3	75	50	150	225	2
20160()	0.00	1975	1993	34.1	65	85	150	450	2
20170()	0.00	1975	1991	28.8	65	85	150	450	2
20200(N)	0.00	1980	1993	48.8	50	155	150	375	2
20200(S)	0.00	1980	1993	45.9	50	155	150	375	2
20650()	0.00	1979	1993	39.6	65	115	150	450	5
20650()	13.20	1979	1989	30.1	65	115	150	450	5
20751()	0.00	1982	1993	39.9		120	150	375	5
26420()	1.60	1980	1990	27.5		40	150	150	2
26420()	10.90	1974	1990	13.9		60	150	225	2
27060()	0.00	1970	1989	23.1	70	80	150	300	2
28245()	0.00	1978	1992	12.4	40	80	0	300	2
28245()	4.00	1978	1992	37.5	40	70	150	150	2
29600()	0.00	1974	1992	33.1	60	80	150	300	2
30020()	0.00	1975	1993	20.9	60	90	150	400	2
30080()	0.00	1974	1993	37.2	65	60	150	300	2
31120()	0.00	1975	1992	23.4		40	150	375	2
33200()	0.00	1981	1993	38.7	50	40	150	300	5
33250()	0.00	1984	1992	12.8	50	100	150	150	2
33250()	1.60	1984	1992	25.5	40	60	150	150	2
44720()	0.00	1976	1992	18.9		40	150	450	2
45420()	0.00	1973	1992	39.9	80	90	150	450	2
46280()	0.00	1982	1993	23.3	80	55	150	400	2
51070()	0.30	1986	1993	30.3		40	150	225	3
51070()	11.90	1980	1992	19.6		40	150	225	2
Region4:	NORTHERN REGION SECTIONS								
LHRS	OFFSET	Begin Yr	End Yr	delta PCI	Overlay	AC	GB	SB	sbgd
17030(N)	0.00	1971	1990	17.7		145	150	600	2
17030(S)	0.00	1971	1990	18.8	70	145	150	300	2
17060(N)	0.00	1975	1993	30.8		140	150	450	1
17060(N)	10.30	1976	1993	37.8		115	150	450	2
17060(S)	10.30	1976	1993	34.1		140	150	450	2
28010()	0.00	1972	1990	19.0		40	150	150	1
33330()	0.00	1980	1993	42.8		50	150	450	1
33330()	9.80	1977	1993	36.1		65	150	600	2
35380()	0.00	1979	1993	40.5		140	150	150	2
40200()	38.20	1984	1992	37.7		40	150	300	1
42500()	0.00	1972	1990	26.7		40	150	150	2
46480()	3.60	1974	1993	23.9	23	40	150	600	1+
67800()	1.30	1978	1993	32.0		60	150	300	2

Region5:	NORTHWESTERN REGION SECTIONS								
LHRS	OFFSET	Begin Yr	End Yr	delta PCI	Overlay	AC	GB	SB	sbgd
18170()	10.40	1979	1992	32.0		40	150	300	1+
18170()	21.70	1980	1992	26.9		50	150	450	2
21360()	10.70	1977	1991	38.3		65	150	375	5
21554()	0.60	1979	1993	38.6		100	150	450	1+

Notes:

1. The layer thickness of Overlay, Asphalt Concrete (AC), Granular Base (GB), and Subbase (SB) is in mm.
2. Numbers in the column "sbgd" indicates the subgrade type as listed in Table 4.5 ("1" for Granular, "2" for Sandy Silt and Clay Till with silt<40% or very fine sand and silt<45%, and so on. "1+" indicates the rock-cut or rock-fill type of subgrade. Since this type of subgrade is missing in the MTO Pavement Design and Rehabilitation Manual, the highest M_R coefficient in Table 4.5 (79.3 MPa) is used.).

Appendix C

Data Sheet for Cluster Analysis

Regn_Dist_LHRS	delta AGE	delta PCI	He(mm)	Ms(MPa)	Acc.Traf.
SW_3_12900()	11	16.8	514	41.4	422105
SW_3_13595(N)	13	24.6	663	41.4	4119299
SW_3_13595(S)	13	32.5	663	41.4	7139752
SW_3_13605(N)	9	14.6	630	41.4	4838165
SW_3_13605(S)	9	13.0	630	41.4	4523059
SW_3_13620()	12	25.6	530	41.4	2178483
SW_3_13630()	13	27.1	701	41.4	1835143
SW_3_14510()	10	35.6	863	41.4	965526
SW_3_14530()	14	32.5	590	41.4	766977
SW_3_14570()	14	27.5	630	41.4	4990202
SW_3_16220()0.00	5	19.9	470	34.5	636137
SW_3_16220()15.60	13	15.8	790	34.5	1075575
SW_5_16970()	12	34.1	777	41.4	9097864
SW_5_17000(N)	11	18.4	777	41.4	5897502
SW_3_24110()	14	31.7	526	27.6	541532
SW_3_24570()	12	27.5	833	34.5	949883
SW_3_24600()	10	36.7	498	27.6	413909
SW_3_24960()	14	34.5	618	72.4	860802
SW_3_25480()	10	27.4	660	34.5	569509
SW_5_25620()	13	25.4	667	41.4	1276866
SW_5_25950()	12	18.7	797	41.4	2945249
SW_3_33010()	13	27.3	633	34.5	1450663
SW_2_36600()	13	30.0	503	34.5	372921
SW_1_37300()	6	42.5	518	34.5	155062
SW_1_37320()	5	28.8	575	34.5	256505
SW_2_37620()	12	26.4	625	34.5	474398
SW_2_47740(E)	6	22.9	936	72.4	22831468
SW_2_47740(W)	6	25.3	936	72.4	22831468
CR_4_10014(W)	10	34.4	707	34.5	6608322
CR_6_10840()2.10	10	15.2	663	41.4	3485577
CR_6_10840()10.30	10	27.5	680	41.4	3695866
CR_6_10850()	8	25.9	680	41.4	1554329
CR_4_13170()	11	24.5	619	34.5	2400408
CR_4_13180()	10	32.1	839	41.4	893648
CR_6_14330()0.00	13	18.9	590	41.4	1389681
CR_6_14330()2.90	13	19.9	737	41.4	1117712
CR_6_14421()	10	27.2	730	41.4	8434640
CR_6_14460()	13	39.2	755	41.4	6000801
CR_6_16120()	13	34.9	547	34.5	3411333
CR_6_16470()	8	14.9	804	41.4	1689921
CR_6_19380()	10	34.1	641	34.5	2056651
CR_6_31030()	15	21.8	641	34.5	2925680
CR_6_48500(S)	10	14.2	935	41.4	9332929
CR_6_48500(N)	10	17.5	935	41.4	23878398
CR_6_48515(S)	10	13.5	865	41.4	5876513
ER_9_10320()	14	40.9	620	41.4	1200493
ER_9_10330()	14	50.9	620	41.4	280261
ER_9_10340()	14	47.5	599	41.4	148989
ER_9_10360()	12	40.1	489	41.4	274133
ER_7_14900()0.00	9	42.5	700	41.4	266261

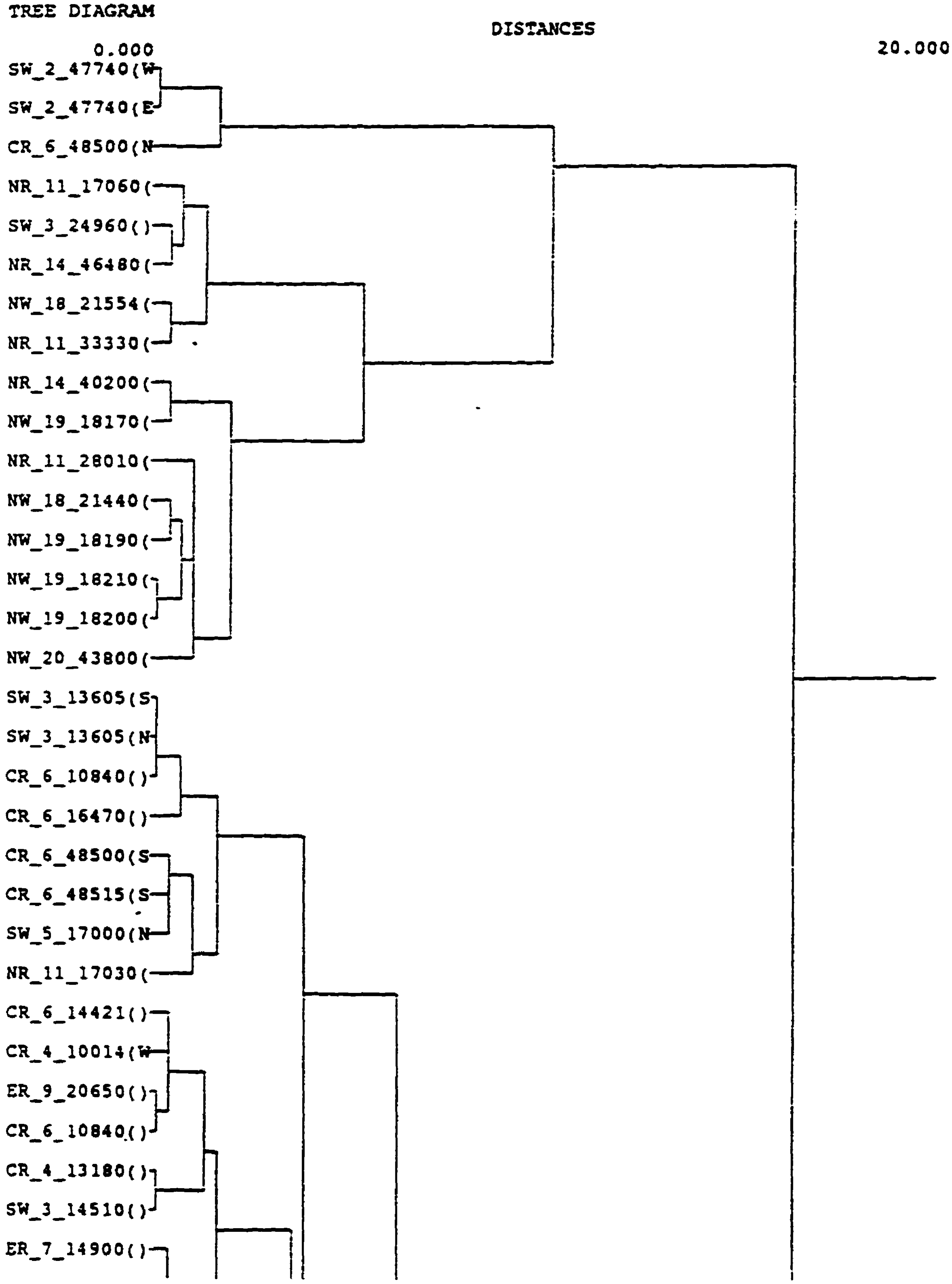
Regn_Dist_LHRS	delta AGE	delta PCI	He(mm)	Ms(MPa)	Acc.Traf.
ER_7_14900()9.00	9	32.0	675	41.4	252803
ER_9_20110()	9	39.8	468	41.4	745722
ER_9_20150()	14	35.3	513	41.4	766977
ER_9_20160()	14	34.1	686	41.4	336862
ER_9_20170()	12	28.8	686	41.4	266481
ER_9_20200(N)	11	48.8	694	41.4	425596
ER_9_20200(S)	11	45.9	694	41.4	488228
ER_9_20650()0.00	14	39.6	724	34.5	4043108
ER_9_20650()13.20	10	30.1	724	34.5	4043108
ER_9_20751()	10	39.9	640	34.5	1288493
ER_10_26420()1.60	8	27.5	330	41.4	43032
ER_10_26420()10.90	11	13.9	420	41.4	68851
ER_9_27060()	10	23.1	590	41.4	2108776
ER_10_28245()0.00	13	12.4	380	41.4	102626
ER_10_28245()4.00	13	37.5	418	41.4	102626
ER_10_29600()	14	33.1	570	41.4	264060
ER_9_30020()	14	20.9	649	41.4	398159
ER_9_30080()	14	37.2	555	41.4	721031
ER_7_31120()	13	23.4	480	41.4	208292
ER_9_33200()	11	38.7	500	34.5	191688
ER_10_33250()0.00	6	12.8	475	41.4	159218
ER_10_33250()1.60	10	25.5	405	41.4	267972
ER_10_44720()	13	18.9	530	41.4	483406
ER_9_45420()	10	39.9	723	41.4	1568905
ER_9_46280()	6	23.3	645	41.4	128546
ER_10_51070()0.30	9	30.3	380	34.5	167238
ER_10_51070()11.90	9	19.6	380	41.4	161370
NR_11_17030(N)	12	17.7	840	41.4	3957310
NR_11_17030(S)	12	18.8	671	41.4	11311446
NR_11_17060(N)0.00	15	30.8	730	72.4	7488512
NR_11_17060(N)10.30	14	37.8	680	41.4	6929399
NR_11_17060(S)10.30	14	34.1	730	41.4	3888873
NR_11_28010()	12	19.0	330	72.4	154138
NR_11_33330()0.00	12	42.8	550	72.4	74328
NR_11_33330()9.80	14	36.1	680	41.4	111003
NR_11_35380()	12	40.5	530	41.4	2013350
NR_14_40200()	7	37.7	430	72.4	74915
NR_11_42500()	12	26.7	330	41.4	56044
NR_14_46480()	15	23.9	646	79.3	198984
NR_14_67800()	13	32.0	470	41.4	603211
NW_19_18170()10.40	8	32.0	430	79.3	728725
NW_19_18170()21.70	9	26.9	550	41.4	367989
NW_18_21360()	12	38.3	530	34.5	1541475
NW_18_21554()	11	38.6	650	79.3	1487644

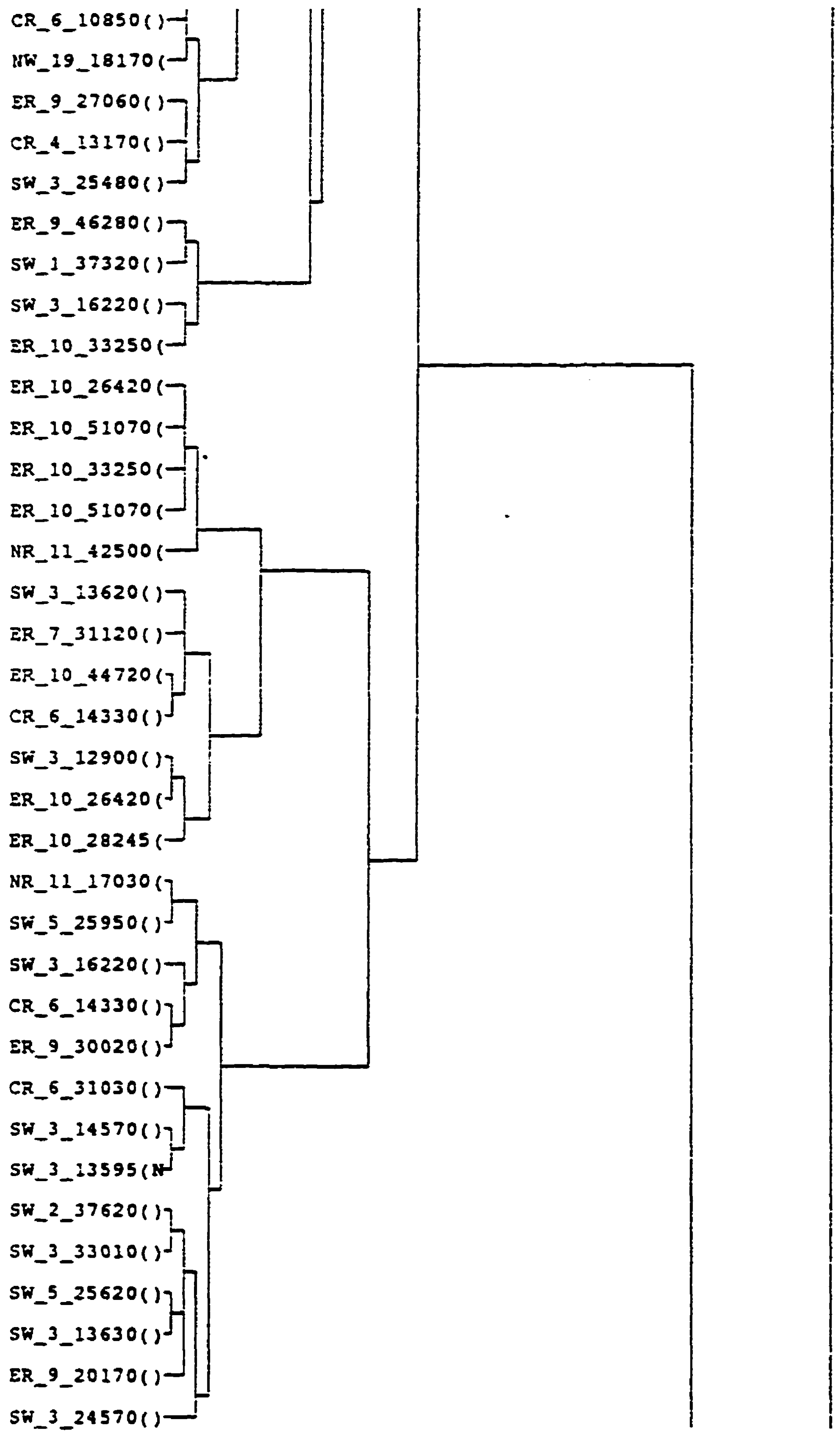
Appendix D

SYSTAT® Tree Plot of Cluster Analysis

DISTANCE METRIC IS EUCLIDEAN DISTANCE
WARD MINIMUM VARIANCE METHOD

TREE DIAGRAM

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SW_3_16220()
ER_10_33250(
ER_10_26420(
ER_10_51070(
ER_10_33250(
ER_10_51070(
NR_11_42500(
SW_3_13620()
ER_7_31120()
ER_10_44720(
CR_6_14330()
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ER_10_26420(
ER_10_28245(
NR_11_17030(
SW_5_25950()
SW_3_16220()
CR_6_14330()
ER_9_30020()
CR_6_31030()
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SW_3_33010()
SW_5_25620()
SW_3_13630()
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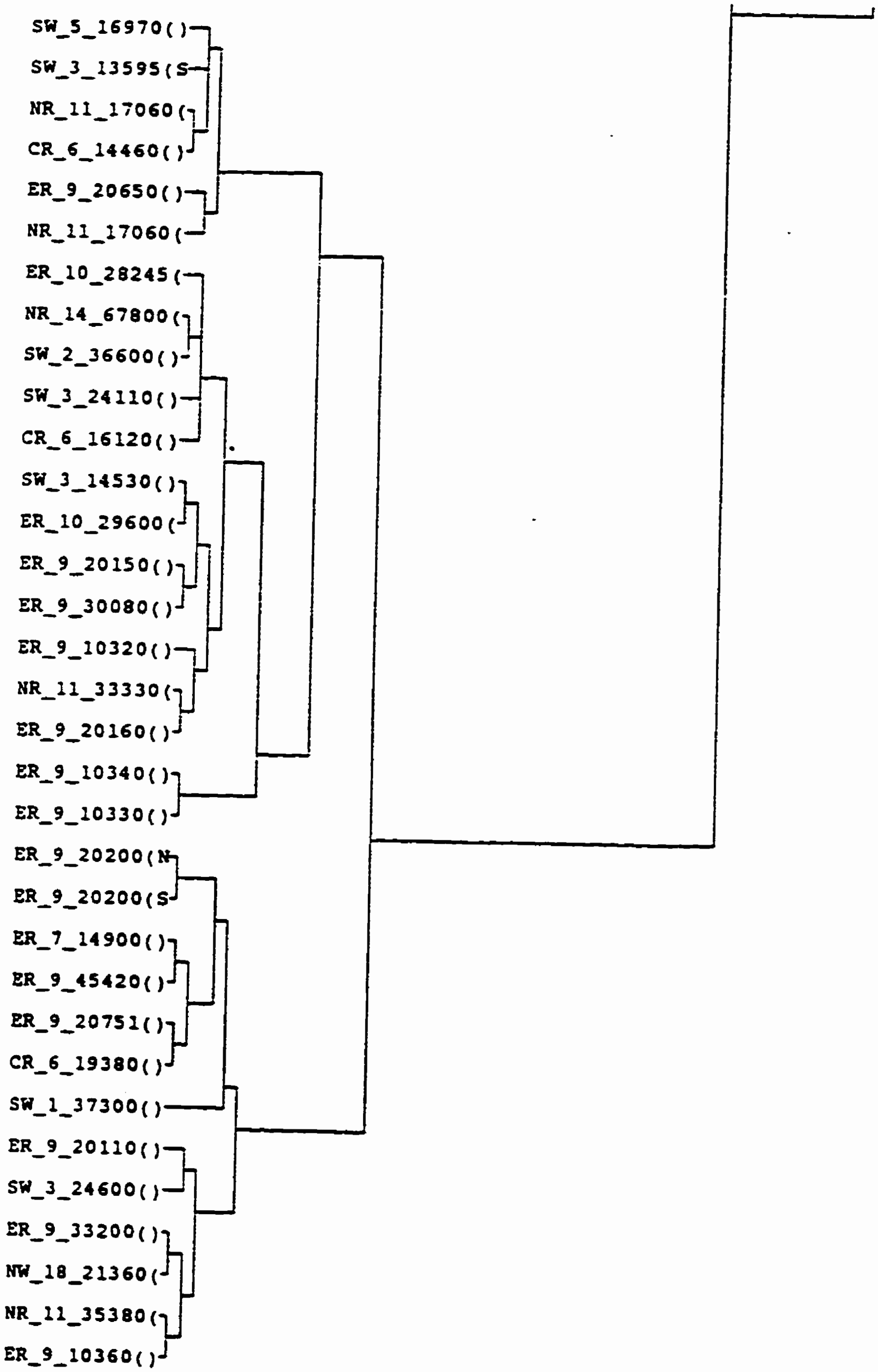
SW_3_24570()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SW_2_36600()
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ER_9_20160()
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ER_9_10330()
ER_9_20200(N
ER_9_20200(S
ER_7_14900()
ER_9_45420()
ER_9_20751()
CR_6_19380()
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ER_9_20110()
SW_3_24600()
ER_9_33200()
NW_18_21360(
NR_11_35380(
ER_9_10360()

Appendix E

SYSTAT® Output of Regression Analysis

SYSTAT OUTPUT FOR CLUSTER "SOUTH_NEW_1"

ITERATION	LOSS	PARAMETER VALUES
0	.3261557D+01	.2362D+01-.6000D-01
1	.3192646D+01	.2362D+01-.6382D-01
2	.3132737D+01	.2751D+01-.5912D-01
3	.3127570D+01	.3411D+01-.4845D-01
4	.3098669D+01	.3301D+01-.5063D-01
5	.3073264D+01	.3406D+01-.5224D-01
6	.3055715D+01	.3906D+01-.4675D-01
7	.3041540D+01	.3989D+01-.4824D-01
8	.3023323D+01	.4407D+01-.4531D-01
9	.3016129D+01	.4879D+01-.4264D-01
10	.3005787D+01	.5061D+01-.4261D-01
11	.2992977D+01	.5800D+01-.4033D-01
12	.2983310D+01	.7201D+01-.3714D-01
13	.2977288D+01	.7421D+01-.3783D-01
14	.2973498D+01	.7876D+01-.3683D-01
15	.2972332D+01	.8735D+01-.3542D-01
16	.2971038D+01	.8571D+01-.3584D-01
17	.2969144D+01	.9092D+01-.3547D-01
18	.2967395D+01	.1003D+02-.3453D-01
19	.2966808D+01	.1087D+02-.3382D-01
20	.2966186D+01	.1112D+02-.3378D-01
21	.2965764D+01	.1239D+02-.3302D-01
22	.2965708D+01	.1237D+02-.3310D-01
23	.2965690D+01	.1268D+02-.3292D-01
24	.2965687D+01	.1268D+02-.3293D-01
25	.2965687D+01	.1271D+02-.3292D-01
26	.2965687D+01	.1272D+02-.3291D-01
27	.2965687D+01	.1272D+02-.3291D-01

DEPENDENT VARIABLE IS PEPO

SOURCE	SUM-OF-SQUARES	DF	MEAN-SQUARE
REGRESSION	28.632321	2	14.316161
RESIDUAL	2.965687	542	0.005472
TOTAL	31.441852	544	
CORRECTED	10.115315	543	

RAW R-SQUARED (1-RESIDUAL/TOTAL) = 0.905677
CORRECTED R-SQUARED (1-RESIDUAL/CORRECTED) = 0.706812

PARAMETER	ESTIMATE	A.S.E.	LOWER	<95%> UPPER
BETA	12.721113	6.008033	0.919227	24.522999
ALPHA	-0.032915	0.003022	-0.038851	-0.026978

ASYMPTOTIC CORRELATION MATRIX OF PARAMETERS

	BETA	ALPHA
BETA	1.000000	
ALPHA	0.982003	1.000000

SYSTAT OUTPUT FOR CLUSTER "NORTH"

ITERATION	LOSS	PARAMETER VALUES
0	.9043026D+00	.2362D+01-.6000D-01
1	.7202758D+00	.2363D+01-.1043D+00
2	.5688370D+00	.2263D+01-.9285D-01
3	.5369005D+00	.2370D+01-.8177D-01
4	.5094649D+00	.2785D+01-.6965D-01
5	.4579669D+00	.3382D+01-.6712D-01
6	.4392778D+00	.3799D+01-.6024D-01
7	.4320253D+00	.4201D+01-.5600D-01
8	.4119959D+00	.4809D+01-.5382D-01
9	.4018562D+00	.6141D+01-.4735D-01
10	.3927163D+00	.6403D+01-.4876D-01
11	.3881706D+00	.7391D+01-.4539D-01
12	.3851868D+00	.8093D+01-.4445D-01
13	.3838991D+00	.8856D+01-.4319D-01
14	.3831326D+00	.9528D+01-.4280D-01
15	.3827459D+00	.1004D+02-.4196D-01
16	.3827002D+00	.1031D+02-.4164D-01
17	.3826731D+00	.1061D+02-.4141D-01
18	.3826716D+00	.1056D+02-.4145D-01
19	.3826715D+00	.1055D+02-.4146D-01
20	.3826715D+00	.1055D+02-.4146D-01
21	.3826715D+00	.1055D+02-.4146D-01
22	.3826715D+00	.1055D+02-.4146D-01

DEPENDENT VARIABLE IS PEPO

SOURCE	SUM-OF-SQUARES	DF	MEAN-SQUARE
REGRESSION	10.007684	2	5.003842
RESIDUAL	0.382672	159	0.002407
TOTAL	10.360383	161	
CORRECTED	2.857170	160	

RAW R-SQUARED (1-RESIDUAL/TOTAL) = 0.963064
CORRECTED R-SQUARED (1-RESIDUAL/CORRECTED) = 0.866066

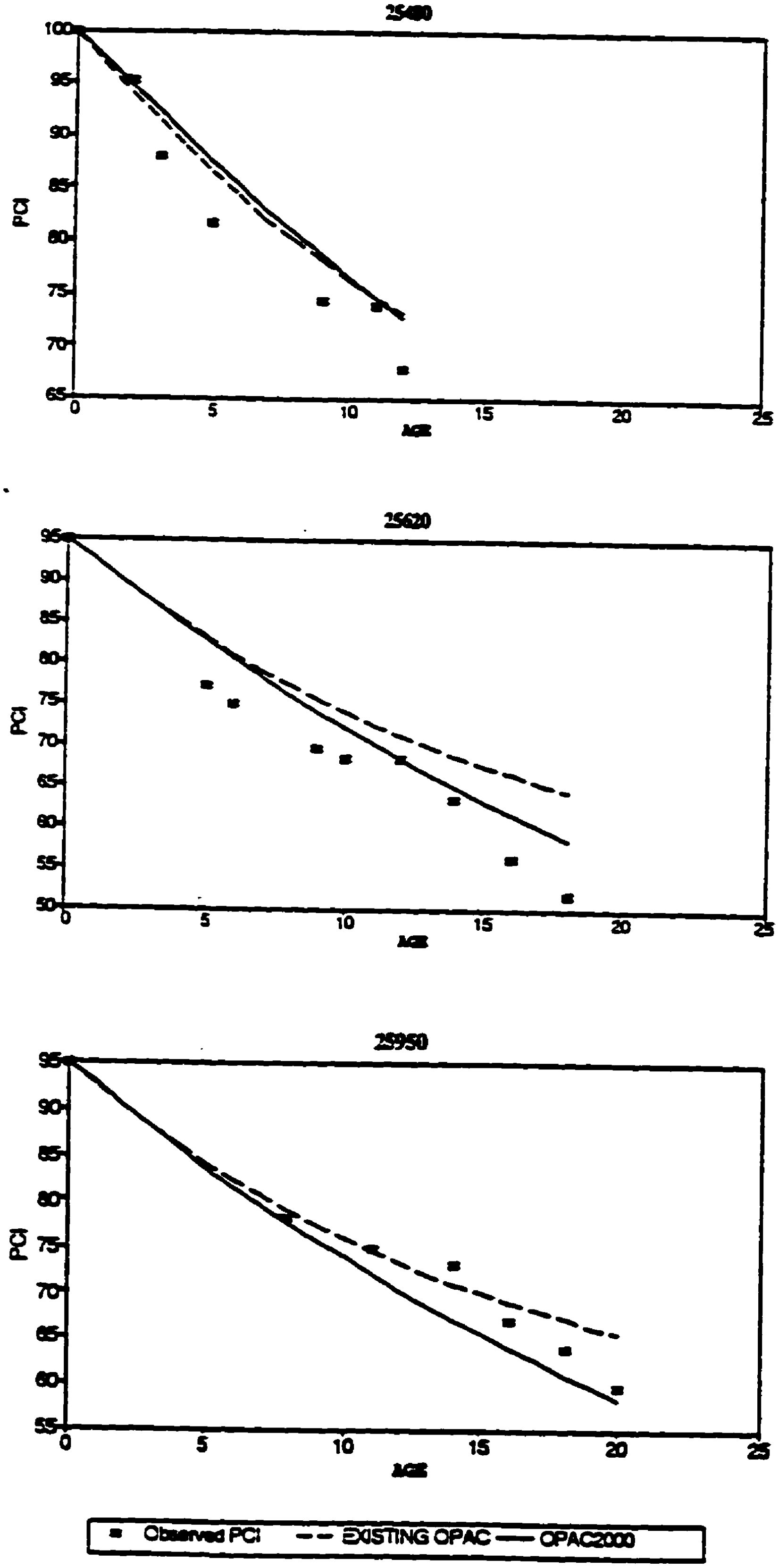
PARAMETER	ESTIMATE	A.S.E.	LOWER	<95%> UPPER
BETA	10.547760	3.049370	4.525266	16.570254
ALPHA	-0.041463	0.003139	-0.047663	-0.035264

ASYMPTOTIC CORRELATION MATRIX OF PARAMETERS

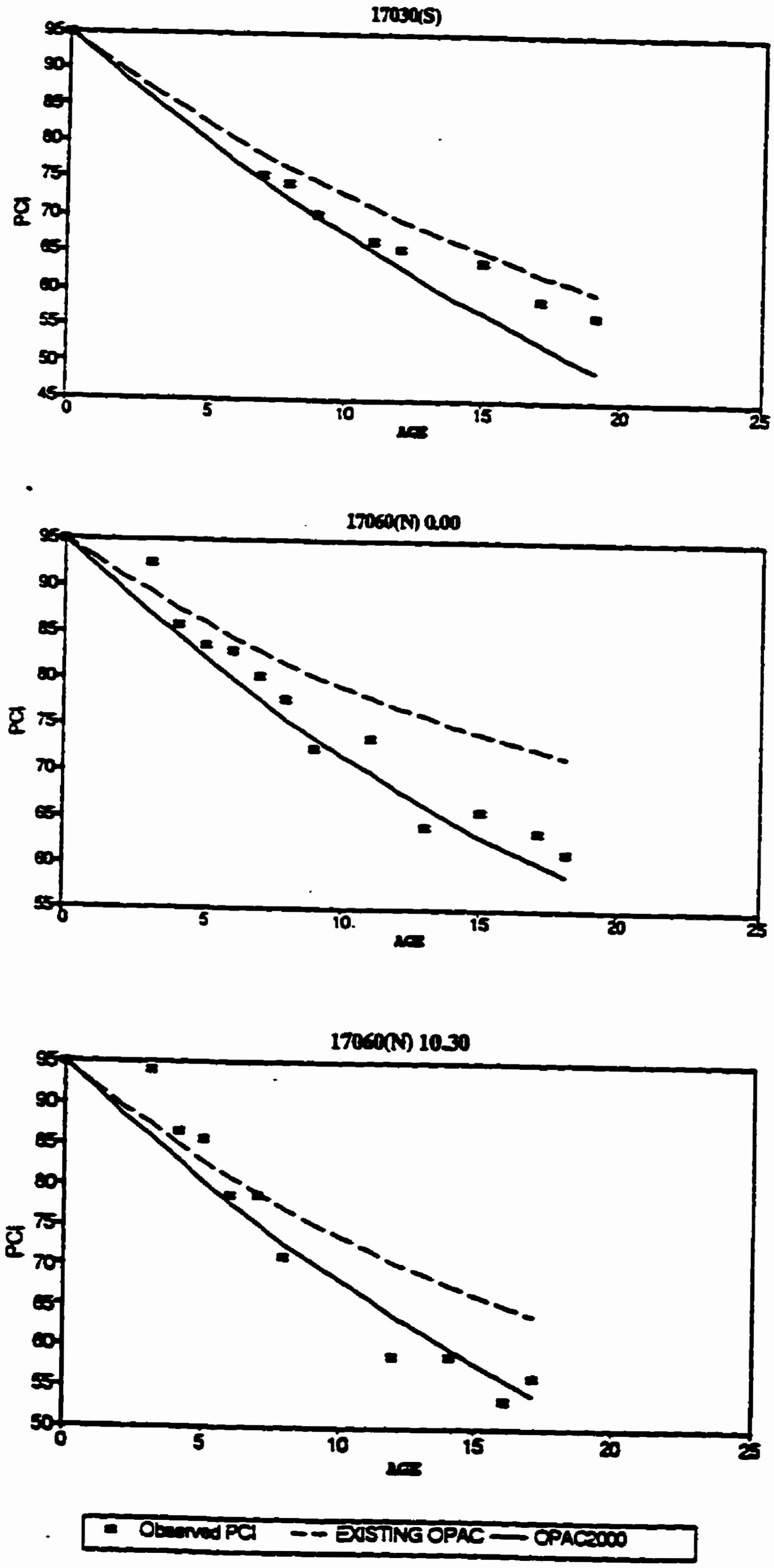
	BETA	ALPHA
BETA	1.000000	
ALPHA	0.963727	1.000000

Appendix F

Sample Pavement Performance Plots



25480
25620
25950
PCI
AGE
Observed PCI
EXISTING OPAC
OPAC2000


17030(S)
PCI
AGE
17060(N) 0.00
PCI
AGE
17060(N) 10.30
PCI
AGE
Observed PCI
EXISTING OPAC
OPAC2000

Appendix G

Partial Derivatives Used in Flexible Pavement Reliability Analysis

(Output of Maple V 3.0)

Partial Derivatives for Reliability Analysis:

$$b1 := .9$$

$$b2 := 344.5$$

$$b3 := 40000$$

$$b4 := 163$$

$$b5 := 2.4455$$

$$b6 := 8.805$$

$$b7 := .37239\ 10^{-5}$$

$$z := \frac{b1\ He\ b2^{1/3}}{Ms^{1/3}}$$

$$Ws := \frac{1}{2}\frac{b3}{Ms^{2/3}\ b1\ He\ b2^{1/3}\sqrt{1+\frac{b4^2\ Ms^{2/3}}{b1^2\ He^2\ b2^{2/3}}}}$$

$$PCI := P0 - \frac{5}{32}\frac{b5\ b7\ b3^6\ N}{Ms^4\ b1^6\ He^6\ b2^2\left(1+\frac{b4^2\ Ms^{2/3}}{b1^2\ He^2\ b2^{2/3}}\right)^3} - \frac{5}{131072}\frac{b6\ b7^3\ b3^{18}\ N^3}{Ms^{12}\ b1^{18}\ He^{18}\ b2^6\left(1+\frac{b4^2\ Ms^{2/3}}{b1^2\ He^2\ b2^{2/3}}\right)^9} - P0\left(1-\frac{1}{1+\frac{1}{2}\frac{\beta\ b3}{Ms^{2/3}\ b1\ He\ b2^{1/3}\sqrt{1+\frac{b4^2\ Ms^{2/3}}{b1^2\ He^2\ b2^{2/3}}}}}\right)\left(1-e^{\alpha AGE}\right)$$

$$\frac{\partial}{\partial P0}P := 1-\left(1-\frac{1}{1+\frac{1}{2}\frac{\beta\ b3}{Ms^{2/3}\ b1\ He\ b2^{1/3}\sqrt{1+\frac{b4^2\ Ms^{2/3}}{b1^2\ He^2\ b2^{2/3}}}}}\right)\left(1-e^{\alpha AGE}\right)$$

$$\frac{\partial}{\partial He} P := \frac{15}{16} \frac{b5\, b7\, b3^6\, N}{Ms^4\, b1^6\, He^7\, b2^2\, \%1^3} - \frac{15}{16} \frac{b5\, b7\, b3^6\, N\, b4^2}{Ms^{10/3}\, b1^8\, He^9\, b2^{8/3}\, \%1^4}$$
$$+ \frac{45}{65536} \frac{b6\, b7^3\, b3^{18}\, N^3}{Ms^{12}\, b1^{18}\, He^{19}\, b2^6\, \%1^9} - \frac{45}{65536} \frac{b6\, b7^3\, b3^{18}\, N^3\, b4^2}{Ms^{34/3}\, b1^{20}\, He^{21}\, b2^{20/3}\, \%1^{10}}$$
$$- \frac{P0 \left(-\frac{1}{2} \frac{\beta\, b3}{Ms^{2/3}\, b1\, He^2\, b2^{1/3} \sqrt{\%1}} + \frac{1}{2} \frac{\beta\, b3\, b4^2}{b1^3\, He^4\, b2\, \%1^{3/2}} \right) \left(1 - e^{\alpha AGE} \right)}{\left(1 + \frac{1}{2} \frac{\beta\, b3}{Ms^{2/3}\, b1\, He\, b2^{1/3} \sqrt{\%1}} \right)^2}$$

$$\%1 := 1 + \frac{b4^2\, Ms^{2/3}}{b1^2\, He^2\, b2^{2/3}}$$

$$\frac{\partial}{\partial Ms} P := \frac{5}{8} \frac{b5\, b7\, b3^6\, N}{Ms^5\, b1^6\, He^6\, b2^2\, \%1^3} + \frac{5}{16} \frac{b5\, b7\, b3^6\, N\, b4^2}{Ms^{13/3}\, b1^8\, He^8\, b2^{8/3}\, \%1^4}$$
$$+ \frac{15}{32768} \frac{b6\, b7^3\, b3^{18}\, N^3}{Ms^{13}\, b1^{18}\, He^{18}\, b2^6\, \%1^9} + \frac{15}{65536} \frac{b6\, b7^3\, b3^{18}\, N^3\, b4^2}{Ms^{37/3}\, b1^{20}\, He^{20}\, b2^{20/3}\, \%1^{10}}$$
$$- \frac{P0 \left(-\frac{1}{3} \frac{\beta\, b3}{Ms^{5/3}\, b1\, He\, b2^{1/3} \sqrt{\%1}} - \frac{1}{6} \frac{\beta\, b3\, b4^2}{Ms\, b1^3\, He^3\, b2\, \%1^{3/2}} \right) \left(1 - e^{\alpha AGE} \right)}{\left(1 + \frac{1}{2} \frac{\beta\, b3}{Ms^{2/3}\, b1\, He\, b2^{1/3} \sqrt{\%1}} \right)^2}$$

$$\%1 := 1 + \frac{b4^2\, Ms^{2/3}}{b1^2\, He^2\, b2^{2/3}}$$

$$\frac{\partial}{\partial N} P := -\frac{5}{32} \frac{b5\, b7\, b3^6}{Ms^4\, b1^6\, He^6\, b2^2 \left(1 + \frac{b4^2\, Ms^{2/3}}{b1^2\, He^2\, b2^{2/3}} \right)^3}$$
$$- \frac{15}{131072} \frac{b6\, b7^3\, b3^{18}\, N^2}{Ms^{12}\, b1^{18}\, He^{18}\, b2^6 \left(1 + \frac{b4^2\, Ms^{2/3}}{b1^2\, He^2\, b2^{2/3}} \right)^9}$$

Appendix H

Results of Effectiveness and Project Costs Calculations

Alternative A effectiveness					
Section	Needs Yr.	No delay	Delay 1Yr.	Delay 2Yr.	Delay 3Yr.
N1	1996	4053378	3040014	2076157	1160076
N2	1994	4425417	3490581	2598765	1746437
N3	1992	12173220	10078047	8085357	6191178
N4	1999	3955084	1843105		
N5	2000	2194998			
N6	1995	3209232	2493031	1808882	1155211
N7	1998	1174373	742223	332366	
N8	1991	2954523	2466082	2000152	1555984
N9	1991	18534258	14080376	9931554	6087984
N10	2001				
N11	1994	1168032	911432	668445	438641
N12	2000	1235700			
N13	1993	13506882	10157417	7065691	4232420
N14	1993	1903621	1553923	1219880	900755
N15	1993	1425964	1158953	904201	661185
N16	1998	3622960	2235806	909463	
N17	1995	7270890	5724549	4250299	2845271

Alternative B effectiveness					
Section	Needs Yr.	No delay	Delay 1Yr.	Delay 2Yr.	Delay 3Yr.
N1	1996	4096596	3083232	2114965	1190064
N2	1994	4626524	3691687	2790474	1919351
N3	1992	12510153	10414981	8412732	6499436
N4	1999	3985493	1873515		
N5	2000	2194998			
N6	1995	3271632	2555431	1866914	1204923
N7	1998	1179087	746937	335424	
N8	1991	3017586	2529145	2061785	1614471
N9	1991	24695584	20241702	15976847	11892188
N10	2001				
N11	1994	1241043	984443	738071	501422
N12	2000	1235700			
N13	1993	16970677	13621212	10437374	7413162
N14	1993	1946640	1596942	1261272	938893
N15	1993	1461947	1194936	938933	693413
N16	1998	3686514	2299360	951967	
N17	1995	7352111	5805770	4326075	2909704

Alternative C effectiveness					
Section	Needs Yr.	No delay	Delay 1Yr.	Delay 2Yr.	Delay 3Yr.
N1	1996	4126878	3113514	2142307	1211232
N2	1994	4725063	3790227	2884180	2003660
N3	1992	12759867	10664694	8654679	6726448
N4	1999	3999146	1887168		
N5	2000	2194998			
N6	1995	3313440	2597239	1905810	1237995
N7	1998	1183419	751268	338354	
N8	1991	3071497	2583056	2114409	1664521
N9	1991	26463146	22009263	17705731	13543717
N10	2001				
N11	1994	1265110	1008510	760887	521883
N12	2000	1235700			
N13	1993	17783373	14433908	11221286	8139504
N14	1993	1981014	1631316	1294324	969403
N15	1993	1482364	1215353	958489	711327
N16	1998	3711611	2324458	968563	
N17	1995	7425619	5879278	4394138	2967784

Cost ($/2lane-km) and Sectional Data					
Section	AADT	Length(km)	Cost A	Cost B	Cost C
N1	4900	6.0	96600	289800	483000
N2	4475	6.0	96600	289800	483000
N3	5800	10.3	165830	497490	829150
N4	5350	11.6	186760	560280	933800
N5	5600	11.6	186760	560280	933800
N6	1300	16.0	257600	772800	1288000
N7	1300	9.8	157780	473340	788900
N8	1300	11.0	177100	531300	885500
N9	6750	19.1	307510	922530	1537550
N10	500	35.6	573160	1719480	2865800
N11	800	9.2	148120	444360	740600
N12	2300	15.9	255990	767970	1279950
N13	9500	10.1	162610	487830	813050
N14	900	11.3	181930	545790	909650
N15	525	14.9	239890	719670	1199450
N16	2300	17.6	283360	850080	1416800
N17	1650	27.5	442750	1328250	2213750

Appendix I

LINDO Integer Programming Outputs

LINDO output for budget level $ 500,000/year

```
MAX      2954523 N8A91 + 3017586 N8B91 + 3071497 N8C91 + 18534258 N9A91
       + 24695584 N9B91 + 26463146 N9C91 + 2466082 N8A92 + 2529145 N8B92
       + 2583056 N8C92 + 14080376 N9A92 + 20241702 N9B92 + 22009264 N9C92
       + 12173220 N3A92 + 12510153 N3B92 + 12759867 N3C92 + 2000152 N8A93
       + 2061785 N8B93 + 2114409 N8C93 + 9931554 N9A93 + 15976847 N9B93
       + 17705732 N9C93 + 10078047 N3A93 + 10414981 N3B93 + 10664694 N3C93
       + 13506882 N13A93 + 16970676 N13B93 + 17783372 N13C93 + 1903621 N14A93
       + 1946640 N14B93 + 1981014 N14C93 + 1425964 N15A93 + 1461947 N15B93
       + 1482364 N15C93 + 1555984 N8A94 + 1614471 N8B94 + 1664521 N8C94
       + 6087984 N9A94 + 11892188 N9B94 + 13543717 N9C94 + 8085357 N3A94
       + 8412732 N3B94 + 8654679 N3C94 + 10157417 N13A94 + 13621212 N13B94
       + 14433908 N13C94 + 1553923 N14A94 + 1596942 N14B94 + 1631316 N14C94
       + 1158953 N15A94 + 1194936 N15B94 + 1215353 N15C94 + 4425417 N2A94
       + 4626524 N2B94 + 4725063 N2C94 + 1168032 N11A94 + 1241043 N11B94
       + 1265110 N11C94 + 6191178 N3A95 + 6499436 N3B95 + 6726448 N3C95
       + 7065691 N13A95 + 10437374 N13B95 + 11221286 N13C95 + 1219880 N14A95
       + 1261272 N14B95 + 1294324 N14C95 + 904201 N15A95 + 938933 N15B95
       + 958489 N15C95 + 3490581 N2A95 + 3691687 N2B95 + 3790227 N2C95
       + 911432 N11A95 + 984443 N11B95 + 1008510 N11C95 + 7270890 N17A95
       + 7352111 N17B95 + 7425619 N17C95 + 3209232 N6A95 + 3271632 N6B95
       + 3313440 N6C95
 SUBJECT TO
     UNQ1)    N8A91 + N8B91 + N8C91 + N8A92 + N8B92 + N8C92 + N8A93 + N8B93
       + N8C93 + N8A94 + N8B94 + N8C94 <=   1
     UNQ2)    N9A91 + N9B91 + N9C91 + N9A92 + N9B92 + N9C92 + N9A93 + N9B93
       + N9C93 + N9A94 + N9B94 + N9C94 <=   1
     UNQ3)    N3A92 + N3B92 + N3C92 + N3A93 + N3B93 + N3C93 + N3A94 + N3B94
       + N3C94 + N3A95 + N3B95 + N3C95 <=   1
     UNQ4)    N13A93 + N13B93 + N13C93 + N13A94 + N13B94 + N13C94 + N13A95
       + N13B95 + N13C95 <=   1
     UNQ5)    N14A93 + N14B93 + N14C93 + N14A94 + N14B94 + N14C94 + N14A95
       + N14B95 + N14C95 <=   1
     UNQ6)    N15A93 + N15B93 + N15C93 + N15A94 + N15B94 + N15C94 + N15A95
       + N15B95 + N15C95 <=   1
     UNQ7)    N2A94 + N2B94 + N2C94 + N2A95 + N2B95 + N2C95 <=   1
     UNQ8)    N11A94 + N11B94 + N11C94 + N11A95 + N11B95 + N11C95 <=   1
     UNQ9)    N17A95 + N17B95 + N17C95 <=   1
    UNQ10)    N6A95 + N6B95 + N6C95 <=   1
   BDGT91)    177100 N8A91 + 531300 N8B91 + 885500 N8C91 + 307510 N9A91
       + 922530 N9B91 + 1537550 N9C91 <=   500000
   BDGT92)    177100 N8A92 + 531300 N8B92 + 885500 N8C92 + 307510 N9A92
       + 922530 N9B92 + 1537550 N9C92 + 165830 N3A92 + 497490 N3B92
       + 829150 N3C92 <=   500000
   BDGT93)    177100 N8A93 + 531300 N8B93 + 885500 N8C93 + 307510 N9A93
       + 922530 N9B93 + 1537550 N9C93 + 165830 N3A93 + 497490 N3B93
       + 829150 N3C93 + 162610 N13A93 + 487830 N13B93 + 813050 N13C93
       + 181930 N14A93 + 545790 N14B93 + 909650 N14C93 + 239890 N15A93
       + 719670 N15B93 + 1199450 N15C93 <=   500000
   BDGT94)    177100 N8A94 + 531300 N8B94 + 885500 N8C94 + 307510 N9A94
       + 922530 N9B94 + 1537550 N9C94 + 165830 N3A94 + 497490 N3B94
       + 829150 N3C94 + 162610 N13A94 + 487830 N13B94 + 813050 N13C94
       + 181930 N14A94 + 545790 N14B94 + 909650 N14C94 + 239890 N15A94
       + 719670 N15B94 + 1199450 N15C94 + 96600 N2A94 + 289800 N2B94
       + 483000 N2C94 + 148120 N11A94 + 444360 N11B94 + 740600 N11C94
       <=   500000
   BDGT95)    165830 N3A95 + 497490 N3B95 + 829150 N3C95 + 162610 N13A95
       + 487830 N13B95 + 813050 N13C95 + 181930 N14A95 + 545790 N14B95
       + 909650 N14C95 + 239890 N15A95 + 719670 N15B95 + 1199450 N15C95
       + 96600 N2A95 + 289800 N2B95 + 483000 N2C95 + 148120 N11A95
       + 444360 N11B95 + 740600 N11C95 + 442750 N17A95 + 1328250 N17B95
       + 2213750 N17C95 + 257600 N6A95 + 772800 N6B95 + 1288000 N6C95
       <=   500000
 END
 INTE    81
```

```
LP OPTIMUM FOUND AT STEP      32
OBJECTIVE VALUE =   68455890.0
FIX ALL VARS.(     5)  WITH RC >    .125840E+08
SET     N9B91 TO <=     0 AT     1, BND=    .6764E+08 TWIN= -.1000E+31     47
SET     N9C91 TO <=     0 AT     2, BND=    .6697E+08 TWIN= -.1000E+31     65
SET     N9C92 TO <=     0 AT     3, BND=    .6684E+08 TWIN= -.1000E+31     74
... ...
(314 lines deleted)
... ...
FLIP    N6A95 TO >=          1 AT     8 WITH BND=    65743490.
SET    N17A95 TO <=     0 AT     9, BND=    .6310E+08 TWIN= -.1000E+31    678
DELETE    N17A95 AT LEVEL     9
DELETE     N6A95 AT LEVEL     8
DELETE     N8C91 AT LEVEL     7
DELETE     N8B91 AT LEVEL     6
DELETE     N3C92 AT LEVEL     5
DELETE     N9B92 AT LEVEL     4
DELETE     N9C92 AT LEVEL     3
DELETE     N9C91 AT LEVEL     2
DELETE     N9B91 AT LEVEL     1
ENUMERATION COMPLETE. BRANCHES=    50 PIVOTS=     678

LAST INTEGER SOLUTION IS THE BEST FOUND
RE-INSTALLING BEST SOLUTION...

       OBJECTIVE FUNCTION VALUE

       1)      65387870.

 VARIABLE          VALUE          REDUCED COST
    N8A91          1.000000    -2954523.000000
    N9A91          1.000000   -18534260.000000
    N3B92          1.000000   -12510150.000000
   N13B93          1.000000   -16970680.000000
   N14A94          1.000000    -1553923.000000
    N2A94          1.000000    -4425417.000000
   N11A94          1.000000    -1168032.000000
   N17A95          1.000000    -7270890.000000


      ROW   SLACK OR SURPLUS     DUAL PRICES

NO. ITERATIONS=      679
BRANCHES=    50 DETERM.=  1.000E    0
```

LINDO output for budget level $ 1,000,000/year

```
MAX     2954523 N8A91 + 3017586 N8B91 + 3071497 N8C91 + 18534258 N9A91
      + 24695584 N9B91 + 26463146 N9C91 + 2466082 N8A92 + 2529145 N8B92
      + 2583056 N8C92 + 14080376 N9A92 + 20241702 N9B92 + 22009264 N9C92
      + 12173220 N3A92 + 12510153 N3B92 + 12759867 N3C92 + 2000152 N8A93
      + 2061785 N8B93 + 2114409 N8C93 + 9931554 N9A93 + 15976847 N9B93
      + 17705732 N9C93 + 10078047 N3A93 + 10414981 N3B93 + 10664694 N3C93
      + 13506882 N13A93 + 16970676 N13B93 + 17783372 N13C93 + 1903621 N14A93
      + 1946640 N14B93 + 1981014 N14C93 + 1425964 N15A93 + 1461947 N15B93
      + 1482364 N15C93 + 1555984 N8A94 + 1614471 N8B94 + 1664521 N8C94
      + 6087984 N9A94 + 11892188 N9B94 + 13543717 N9C94 + 8085357 N3A94
      + 8412732 N3B94 + 8654679 N3C94 + 10157417 N13A94 + 13621212 N13B94
      + 14433908 N13C94 + 1553923 N14A94 + 1596942 N14B94 + 1631316 N14C94
      + 1158953 N15A94 + 1194936 N15B94 + 1215353 N15C94 + 4425417 N2A94
      + 4626524 N2B94 + 4725063 N2C94 + 1168032 N11A94 + 1241043 N11B94
      + 1265110 N11C94 + 6191178 N3A95 + 6499436 N3B95 + 6726448 N3C95
      + 7065691 N13A95 + 10437374 N13B95 + 11221286 N13C95 + 1219880 N14A95
      + 1261272 N14B95 + 1294324 N14C95 + 904201 N15A95 + 938933 N15B95
      + 958489 N15C95 + 3490581 N2A95 + 3691687 N2B95 + 3790227 N2C95
      + 911432 N11A95 + 984443 N11B95 + 1008510 N11C95 + 7270890 N17A95
      + 7352111 N17B95 + 7425619 N17C95 + 3209232 N6A95 + 3271632 N6B95
      + 3313440 N6C95
 SUBJECT TO
     UNQ1)   N8A91 + N8B91 + N8C91 + N8A92 + N8B92 + N8C92 + N8A93 + N8B93
      + N8C93 + N8A94 + N8B94 + N8C94 <=   1
     UNQ2)   N9A91 + N9B91 + N9C91 + N9A92 + N9B92 + N9C92 + N9A93 + N9B93
      + N9C93 + N9A94 + N9B94 + N9C94 <=   1
     UNQ3)   N3A92 + N3B92 + N3C92 + N3A93 + N3B93 + N3C93 + N3A94 + N3B94
      + N3C94 + N3A95 + N3B95 + N3C95 <=   1
     UNQ4)   N13A93 + N13B93 + N13C93 + N13A94 + N13B94 + N13C94 + N13A95
      + N13B95 + N13C95 <=   1
     UNQ5)   N14A93 + N14B93 + N14C93 + N14A94 + N14B94 + N14C94 + N14A95
      + N14B95 + N14C95 <=   1
     UNQ6)   N15A93 + N15B93 + N15C93 + N15A94 + N15B94 + N15C94 + N15A95
      + N15B95 + N15C95 <=   1
     UNQ7)   N2A94 + N2B94 + N2C94 + N2A95 + N2B95 + N2C95 <=   1
     UNQ8)   N11A94 + N11B94 + N11C94 + N11A95 + N11B95 + N11C95 <=   1
     UNQ9)   N17A95 + N17B95 + N17C95 <=   1
    UNQ10)   N6A95 + N6B95 + N6C95 <=   1
   BDGT91)   177100 N8A91 + 531300 N8B91 + 885500 N8C91 + 307510 N9A91
      + 922530 N9B91 + 1537550 N9C91 <=   1000000
   BDGT92)   177100 N8A92 + 531300 N8B92 + 885500 N8C92 + 307510 N9A92
      + 922530 N9B92 + 1537550 N9C92 + 165830 N3A92 + 497490 N3B92
      + 829150 N3C92 <=   1000000
   BDGT93)   177100 N8A93 + 531300 N8B93 + 885500 N8C93 + 307510 N9A93
      + 922530 N9B93 + 1537550 N9C93 + 165830 N3A93 + 497490 N3B93
      + 829150 N3C93 + 162610 N13A93 + 487830 N13B93 + 813050 N13C93
      + 181930 N14A93 + 545790 N14B93 + 909650 N14C93 + 239890 N15A93
      + 719670 N15B93 + 1199450 N15C93 <=   1000000
   BDGT94)   177100 N8A94 + 531300 N8B94 + 885500 N8C94 + 307510 N9A94
      + 922530 N9B94 + 1537550 N9C94 + 165830 N3A94 + 497490 N3B94
      + 829150 N3C94 + 162610 N13A94 + 487830 N13B94 + 813050 N13C94
      + 181930 N14A94 + 545790 N14B94 + 909650 N14C94 + 239890 N15A94
      + 719670 N15B94 + 1199450 N15C94 + 96600 N2A94 + 289800 N2B94
      + 483000 N2C94 + 148120 N11A94 + 444360 N11B94 + 740600 N11C94
      <=   1000000
   BDGT95)   165830 N3A95 + 497490 N3B95 + 829150 N3C95 + 162610 N13A95
      + 487830 N13B95 + 813050 N13C95 + 181930 N14A95 + 545790 N14B95
      + 909650 N14C95 + 239890 N15A95 + 719670 N15B95 + 1199450 N15C95
      + 96600 N2A95 + 289800 N2B95 + 483000 N2C95 + 148120 N11A95
      + 444360 N11B95 + 740600 N11C95 + 442750 N17A95 + 1328250 N17B95
      + 2213750 N17C95 + 257600 N6A95 + 772800 N6B95 + 1288000 N6C95
      <=   1000000
 END
 INTE    81
```

```
LP OPTIMUM FOUND AT STEP      45
OBJECTIVE VALUE =    77441940.0
SET     N6B95 TO >=     1 AT     1, BND=    .7393E+08 TWIN=  .7744E+08       49
SET    N17A95 TO <=     0 AT     2, BND=    .7145E+08 TWIN= -.1000E+31       52
SET    N17B95 TO <=     0 AT     3, BND=    .7096E+08 TWIN= -.1000E+31       55
... ...
(2123 lines deleted)
... ...
DELETE    N17C95 AT LEVEL      4
DELETE    N17B95 AT LEVEL      3
DELETE     N6C95 AT LEVEL      2
DELETE     N6B95 AT LEVEL      1
ENUMERATION COMPLETE. BRANCHES=   222 PIVOTS=    3925

LAST INTEGER SOLUTION IS THE BEST FOUND
RE-INSTALLING BEST SOLUTION...

       OBJECTIVE FUNCTION VALUE

       1)      76890980.

 VARIABLE        VALUE          REDUCED COST
    N9B91         1.000000  -24695580.000000
    N8A92         1.000000   -2466082.000000
    N3B92         1.000000  -12510150.000000
   N13C93         1.000000  -17783370.000000
   N14A93         1.000000   -1903621.000000
   N15A94         1.000000   -1158953.000000
    N2C94         1.000000   -4725063.000000
   N11A94         1.000000   -1168032.000000
   N17A95         1.000000   -7270890.000000
    N6A95         1.000000   -3209232.000000

      ROW   SLACK OR SURPLUS     DUAL PRICES

NO. ITERATIONS=    3928
BRANCHES=  222 DETERM.=  1.000E    0
```

Appendix J

OPAC 2000 User's Guide

(Adapted from ITX Stanley, December 1996)

ITX Stanley

February 10, 1997

Mr. Zhiwei He
219-155 University Avenue W.
Waterloo, Ontario
N2L 3E5

Reference: OPAC 2000 USER'S GUIDE

Please consider this as our written permission to use information from ITX Stanley Ltd.'s document entitled "OPAC 200 User's Guide - Dec. 1996" for your Ph.D. thesis at the University of Waterloo.

Sincerely,

ITX Stanley Ltd.

Frank Meyer, Ph.D., P.Eng.
Vice President

FM/ch

ITX Stanley Ltd. 152 Main Street Cambridge ON Canada N1R 6R1 Ph: (519) 622-3005 Fax: (519) 622-2580

1.0 Introduction

About OPAC2000

OPAC2000 is a project-level pavement management tool for pavement design. It incorporates structural analysis and life-cycle cost analysis to evaluate project design alternatives.
OPAC2000 was developed as an enhanced version of the existing OPAC. The Ontario Pavement Analysis of Costs (OPAC) has been used extensively by the Ministry of Transportation of Ontario for about 20 years. Due to significant changes in pavement design needs since its original inception, OPAC was updated in the following areas:

- improved flexible pavement performance prediction model,
- addition of a rigid pavement design module,
- addition of an overlay pavement design module,
- incorporation of reliability analysis,
- improved economic analysis module,
- addition of emission effect model, and
- extension from a DOS system to a Windows© based system.

The engineering and model development for this enhanced pavement design package has been carried out at the University of Waterloo. Software for the package was prepared by Pavement Management Systems Limited, under contract to The University.

System Requirements

The following is a list of the **minimum** hardware requirements to install and run OPAC2000:

CPU:	80486
RAM:	8 Mb of extended or expanded memory
VIDEO:	VA adapter card and color monitor
HARD DISK:	20 Mb of free disk space, this requirement may increase as the database grows
FLOPPY:	3.5"
MOUSE:	Microsoft compatible

Ease of use and speed of processing will increase with the addition of the following **recommended hardware upgrades** to the base configuration listed above:

CPU:	Pentium or better
RAM:	The most beneficial hardware addition that can be made is to add as much RAM as possible. 16 Mb is recommended, although 32 Mb will further improve performance.

This version of the software is currently designed to operate with the following operating system installation:

- DOS Version 5.x or higher
- Windows 3.X, or Windows for Workgroups 3.x

Other versions of the software will run on Windows NT or Windows 95. If your operating system of preference is either of these systems please phone the number listed below (Technical Support) to receive a free upgrade.

Technical Support

If you have any questions about OPAC2000 review the documentation and search the on-line help. If you cannot find the answer contact a Pavement Management Systems representative at:

152 Main Street,
Cambridge, ON
Canada, N1R 6R1
Phone: 519-622-3005
FAX: 519-622-2580

Note, however, that system support for OPAC 2000 has not yet been arranged at the time of preparation of this User's Guide (December, 1996). Consequently, questions and consultations may involve a fee.

Disclaimer

OPAC 2000 is intended to aid personnel who are knowledgeable in pavement engineering, and it cannot replace the professional judgment of a pavement design engineer. The parties and individuals associated in developing the program cannot assume responsibilities for any improper use of the program, or for the accuracy of the sources upon which the program is based.
Information in this document is subject to change without notice and does not represent a commitment on the part of Pavement Management Systems Ltd. The software and/or databases may be used or copied only in accordance with the terms of the agreement. The purchaser may make one copy of the software for backup purposes. No part of this manual and/or databases may be reproduced or transmitted in any form or by any means, electronic or mechanical, including photocopying, recording, or information storage and retrieval systems, for any purpose other than the purchaser's personal use, without the written permission of Pavement Management Systems.

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Unless otherwise noted, all names of streets and persons contained herein are part of a completely fictitious scenario or scenarios and are designed solely to document the use of a Pavement Management System product.

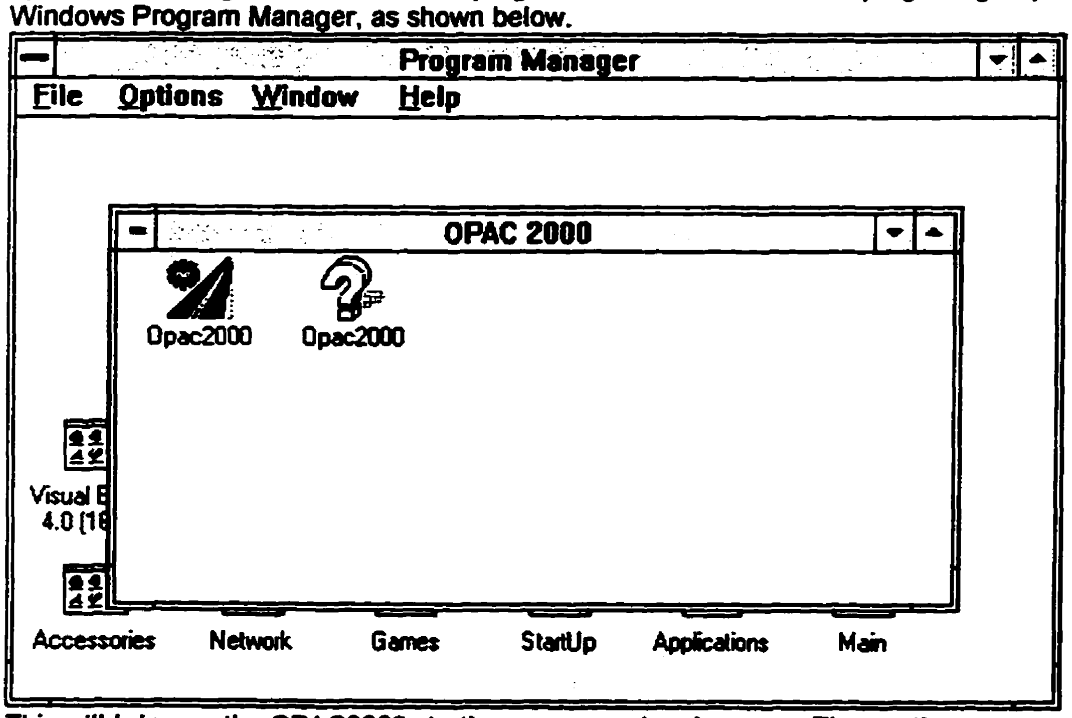
2.0 Starting the Program

Installation

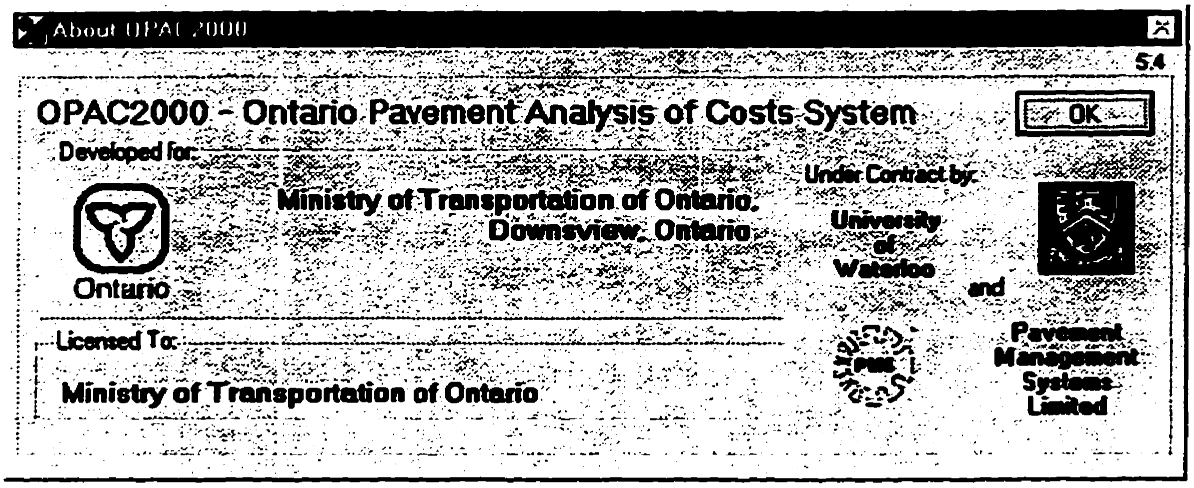
1. Start Microsoft Windows
2. Insert Disk 1 - Setup in drive A: (or B:)
3. From Program Manager, select File menu and choose Run.
4. Type a:\setup (or b:\setup) and press Enter.
5. Follow the setup instructions as they appear on the screen.
6. Restart Windows after the installation is complete.

Starting OPAC2000

Once the OPAC2000 application has been properly installed as described above, it may be started by double-clicking on the OPAC2000 program item in the OPAC2000 program group of the Windows Program Manager, as shown below.



This will bring up the OPAC2000 starting screen and main menu. The starting screen shows the agency/company name of the licensed user as well as provides some information regarding the authors of the software. The starting screen is shown below.



Using the Windows© Interface

The windows interface is designed to make user interaction easy to learn and easy to use. The power of Windows© is in its Graphical User Interface (GUI) - the user can edit data and highlight entries through a point and click approach. This is done by manipulating the Mouse and pressing the Mouse Buttons when input is required. The interface is made up primarily of menus and dialogs .

Menus

Selecting a menu can cause one of two actions to occur - a pulldown menu can be triggered from which further menu options can be made or a dialog can appear. Pulldown menus are sub-groupings of menu options that can lead to further groupings for several levels deep until a dialog is encountered. If a dialog follows a menu item, an ellipsis (...) will be shown to the right of the menu option.

Menu Availability

Menu options are not always available for selection depending upon the applications current status. For example, under the File menu, if no project is currently selected then the Delete Current Project menu option will be disabled.

Select a Menu Using the Mouse

To make a menu selection using the mouse place the pointer over the item and click. In most cases a pulldown menu appears. Click on the desired option to choose it. Releasing the button will initiate the action associated with the highlighted menu item. Click anywhere outside a menu pulldown to close it and not select an option.

Dialogs

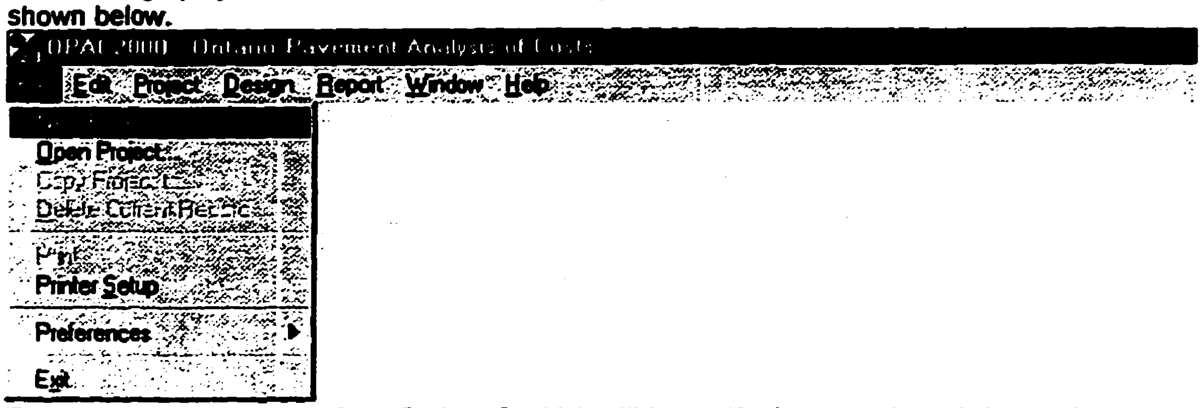
The majority of the user interface is composed of dialogs. Dialogs are rectangular regions on the screen enclosed by a border. In addition to displaying information, a dialog can contain many control objects which allow you to make selections. These control objects include radio buttons, check boxes, popups, list boxes, text boxes, and push buttons.

Managing Dialogs and Windows

A dialog is a type of window. The majority of dialogs are capable of being moved by dragging the title bar to the desired location. In some instances windows can be sized, closed, maximized and/or minimized. When a window is capable of being manipulated as mentioned, its border will have special symbols that are used to indicate each type of action. These symbols vary with each version of Windows© - consult your Windows© documentation for further information.

Starting a New Project

A new design project can be initiated by selecting File, New Project... from the main menu, as shown below.

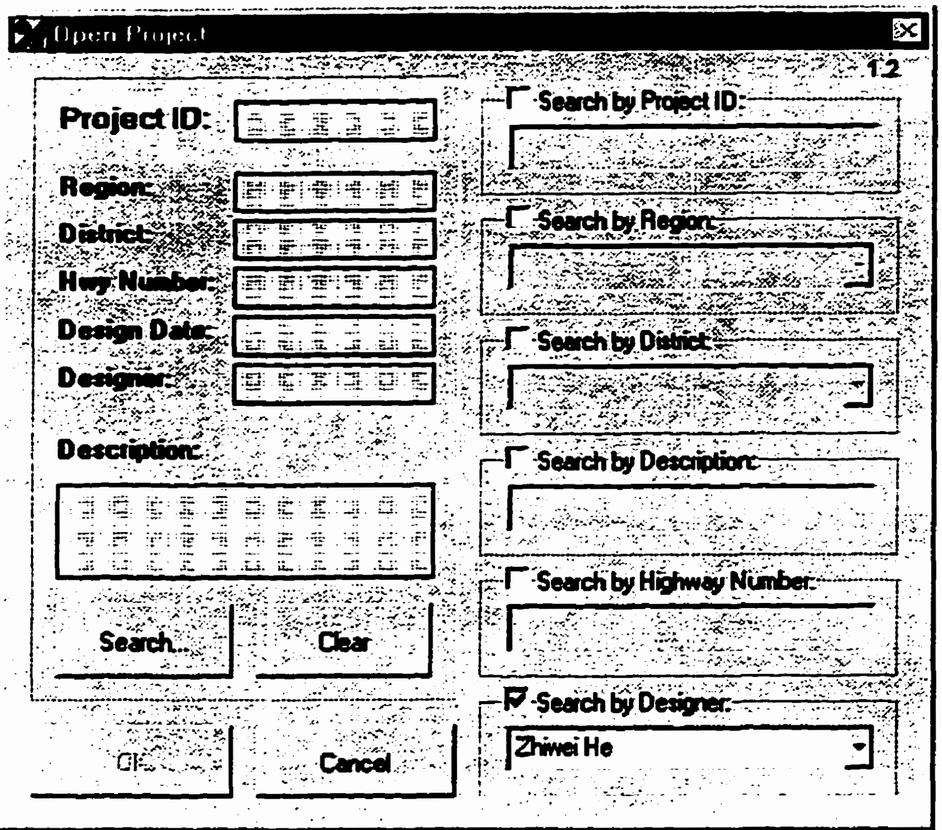


The user will be prompted for a Project ID which will be verified against the existing project database, to ensure that the ID is unique. If it is unique the Section Information input form will be displayed. If it is not unique, the user will warned and then prompted to enter another ID or return to the main menu.

Opening an Existing Project

An existing project can be opened by selecting File, Open Project... from the main menu, as shown below.

A search form will appear which enables the user to build a project search based on a number of criteria as shown below. Once the criteria has been identified, the user can initiate the search. The results of the search will be displayed in a list. The user can then select the project to be opened. Once the project has been selected, the Section Information input form will be displayed with the data from the current project.



If a project is already open, it will be closed when a different existing project is selected as current.

Saving and Deleting Projects

The saving of project and design information is taken care of at the input form level. Whenever a change is made in the input, the user will be prompted as to whether or not they would like to save the data. At this point they are given the option of saving or discarding the data. Because of this feature it is not necessary to 'manually' save the data, i.e., there is no 'Save' item in the File menu that must be clicked before exiting the program.

The current project may be saved under a different Project ID. Saving under a different Project ID allows a project's input data to be duplicated for easy formulation of testing alternatives. Saving under a different Project ID is done by selecting File, Copy Project to... Once again, the user is prompted for a Project ID, and as before, this ID must be unique.

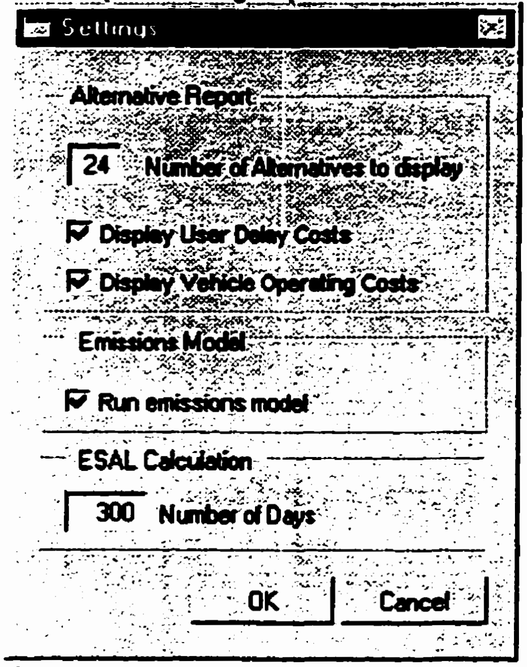
The current project may be deleted by selecting File, Delete Current Project... from the main menu. This deletes all data records associated with the current project, including results. For both Copy Project to... and Delete Current Project... a project must be current, meaning that an existing project must have been opened or a new project created.

Program Preferences and Settings

Preferences may be globally set for the operation of the program by selecting File, Preferences from the main menu. From this point the Data Library, the Layer Names, and the Program Settings can be accessed. In the Data Library, the Material Table and the Maintenance Activity are user defined libraries that describe the materials and maintenance activities that are available to the user during pavement design. The Truck Factors, Lane Distribution Factors, the lane and shoulder width as well as the District List can also be edited at this point.

The Layer Names list identifies the different names that can be used when labeling each of the layers in the structure design. It is very important for the proper operation of the application that one of these layers be named 'Subbase'.

The Report Settings input screen is shown below.



This screen enables the user to turn various economic models on and off, and to identify the number of days per year to use in the ESAL calculation as well as the default number of maximum alternatives to display in the Design Alternatives Report.

3.0 Project Information

The data which defines the design project is divided into 2 parts: Project Information and Design Information. The Project Information contains data such as performance criteria, section identification, project identification, economic, and traffic information. This information can be accessed through the Main Menu by clicking on Project.

The 'Project' section of the main menu reveals the following 3 menu items:

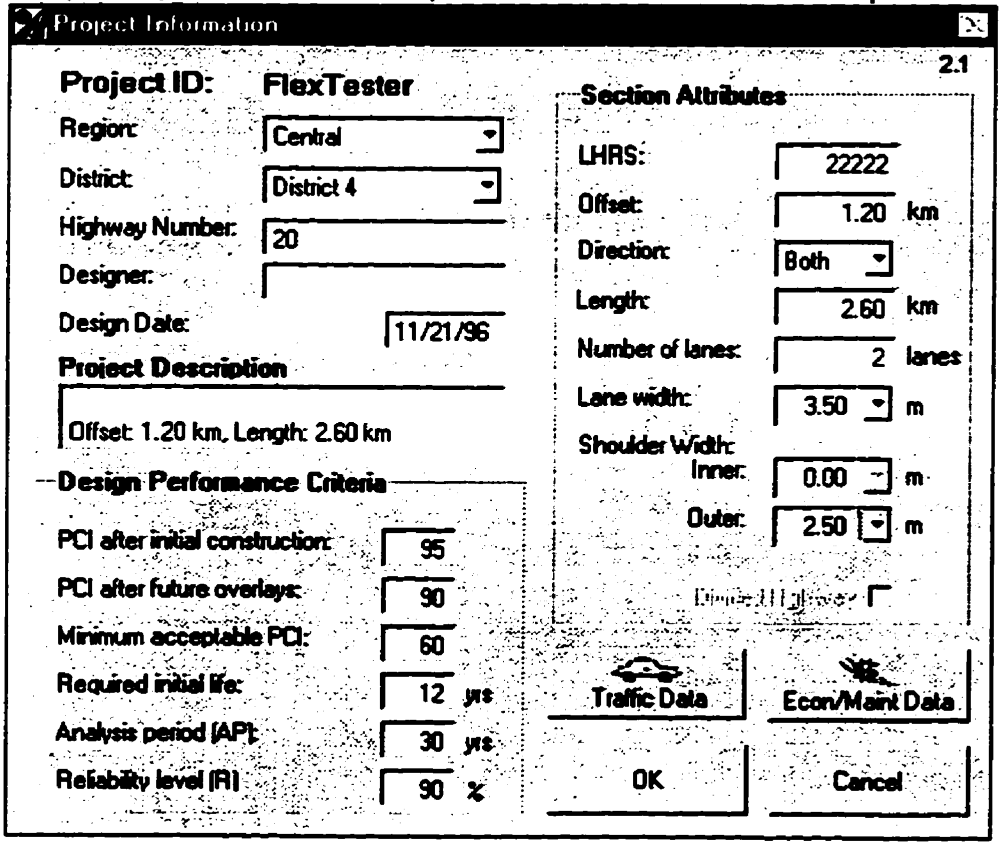
<u>Section Information</u>

<u>Traffic Information</u>

<u>Economic Information</u>

Section Information

The section information screen provides a form for the user to input data such as Project ID, Design Date, LHRS, Performance Criteria, and Geometric information. The input form is shown below:



Project ID

Used to identify the project.

Region

The geographically defined region in which the project is located.

Designer

Enter the name of the project designer/engineer. This can be used later for retrieving specific projects.

Initial Life

Minimum required initial life of pavement to the first overlay.

Analysis Period

The period through which the pavement design will be analyzed. 30 years is suggested for flexible pavement designs, 40 years for rigid pavement designs.

Reliability

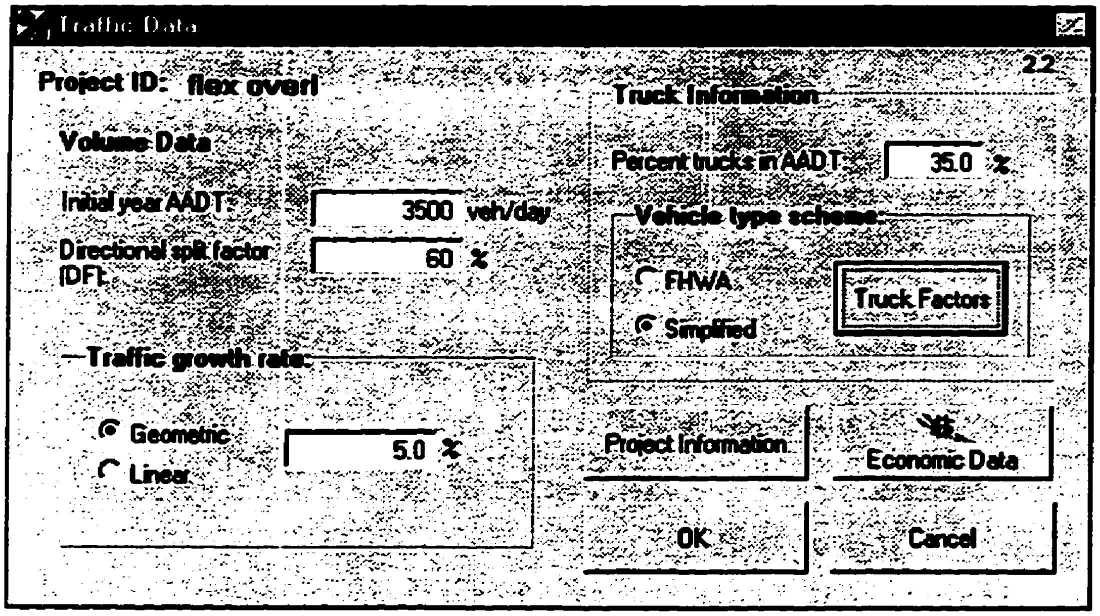
	Urban	Rural
Functional Class	Range (%)	Range (%)
Interstate and Freeways	85.0 - 99.9	80.0-99.9
Principle Arterials	80.0 - 99.0	75.0 - 95.0
Collectors	80.0 - 95.0	75.0 - 95.0
Local	50.0 - 80.0	50.0 - 80.0

Number of Lanes

Total number of traffic lanes in both directions.

Traffic Information

The traffic information screen provides a form for the user to input data such as initial AADT, traffic growth rate, and truck percentages. The input form is shown below:



Lane Distribution Factor (LDF)

Distributes traffic to the lanes according to the AADT and the number of lanes. This is done automatically by OPAC 2000 according to the following table:

Lane Distribution Factor

NUMBER OF LANES IN ONE DIRECTION	AADT	LDF
1	all	1.00
2	<15,000	0.90
	>15,000	0.80
3	<25,000	0.85
	25,000 - 40,000	0.80
	>40,000	0.70
4	<40,000	0.80
	>40,000	0.70

Truck Percent (T%) and Truck Factor (TF)

There are 2 vehicle classification schemes available. These are the FHWA vehicle classification and the simplified truck classification. In both cases the truck traffic is divided into a number of different classifications and a Truck Factor (TF) assigned to each. A percent distribution must be entered to indicate what share of the total truck traffic each class represents. The classifications and their typical Truck Factor's are shown below in the format that is presented in OPAC2000. Truck percent (T%) is the percentage of trucks in the AADT.

FHWA Vehicle Classification

Federal Highway Administration vehicle classification breaks vehicles into 13 different classes. The last 10 of these classes are truck classifications.

FHWA Vehicle Scheme - Truck Factors

Enter/edit truck factor and percent distribution:

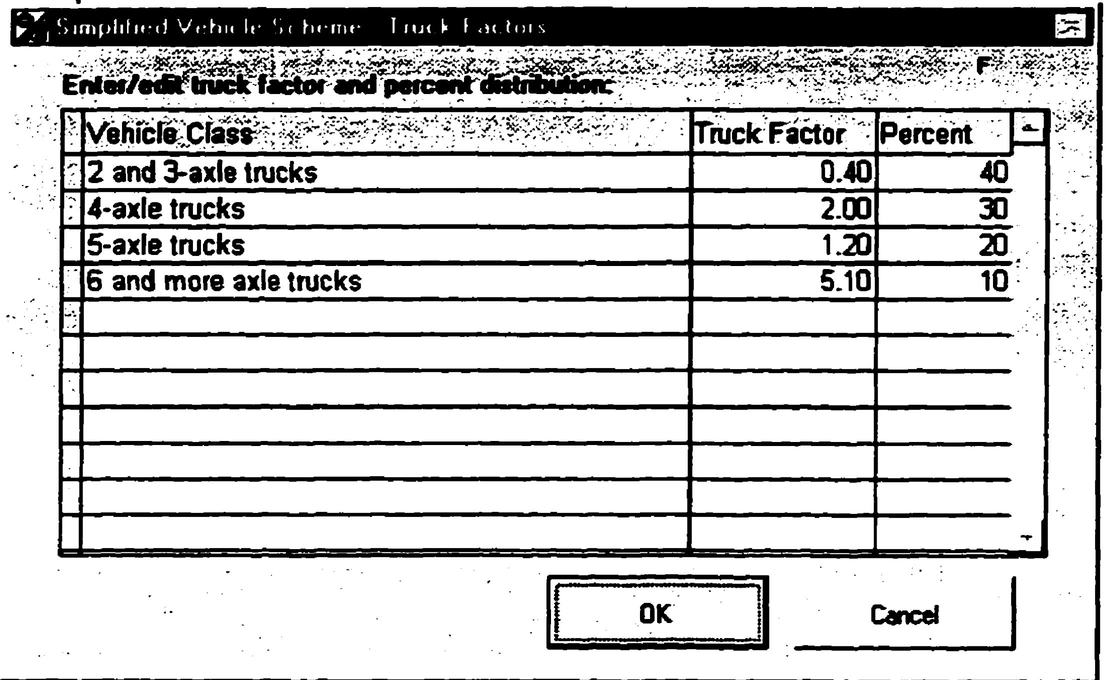
Vehicle Class	Truck Factor	Percent
Buses with two or more axles	1.10	10
Two-axle, six-tire, single unit trucks	0.30	20
Three-axle single unit trucks	0.80	30
Four or more axle single unit trucks	4.00	10
Four or less axle single trailer trucks	0.50	10
Five-axle single trailer trucks	1.20	6
Six or more axle single trailer trucks	3.50	4
Five or less axle multi-trailer trucks	1.50	4
Six-axle multi-trailer trucks	5.10	3
Seven or more axle multi-trailer trucks	4.10	3

OK Cancel

It is important that the sum of the percentage numbers of all truck classes is 100.

Simplified Truck Classification

A simplification of the FHWA vehicle classification which divides truck traffic into 4 vehicle classes.


Simplified Vehicle Scheme - Truck Factors

Enter/edit truck factor and percent distribution:

Vehicle Class	Truck Factor	Percent
2 and 3-axle trucks	0.40	40
4-axle trucks	2.00	30
5-axle trucks	1.20	20
6 and more axle trucks	5.10	10

OK Cancel

It is important that the sum of the percentage numbers of all truck classes is 100.

Initial Year AADT

Average annual daily traffic during the design year.

Directional Split Factor (DF)

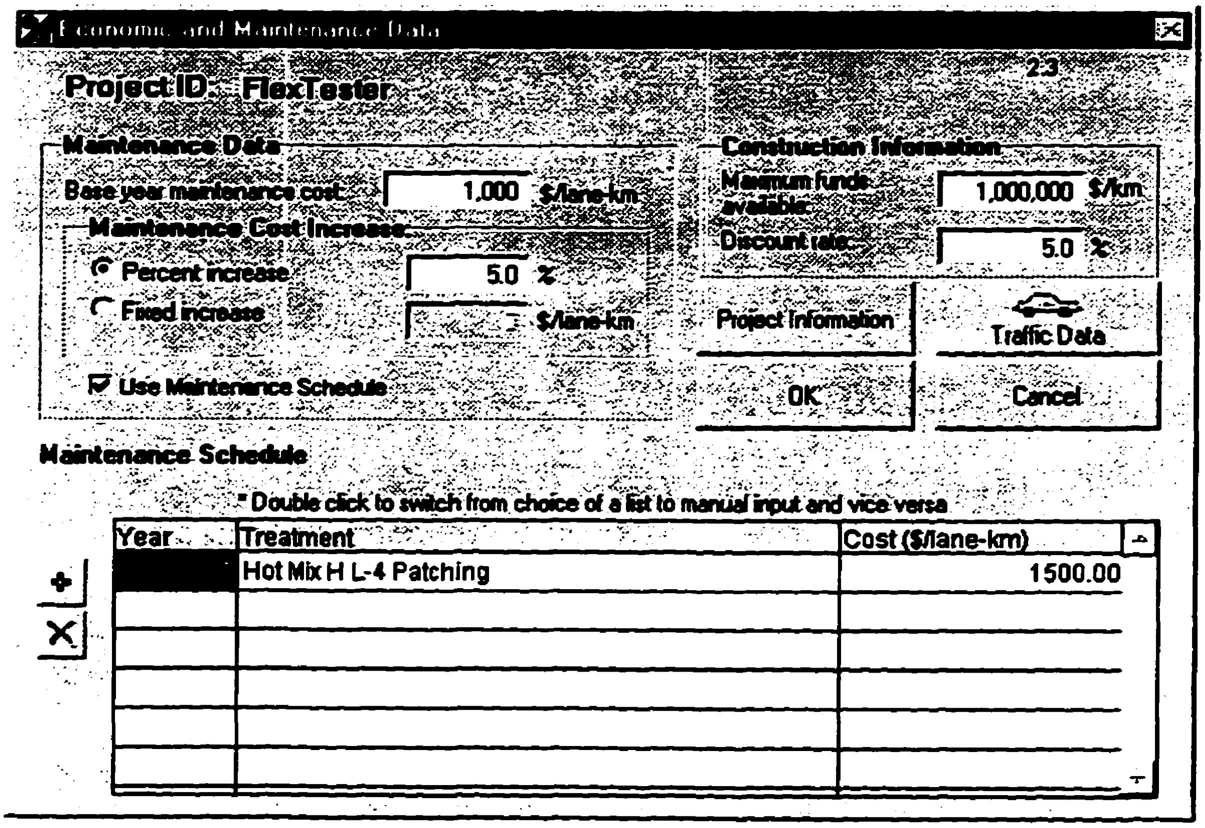
Used for converting two-way traffic into one-way traffic.

Growth Rate (GR)

Annual traffic growth rate. The growth rate can be linear or geometric (similar to compound interest).

Economic Information

The economic information screen provides a form for the user to input data such as initial construction cost, maximum funds available, and maintenance costs. The input form is shown below:



Base Year Maintenance Cost

Expected cost of maintenance during the design year in $/lane-km.

Maintenance Cost Increase

The expected increase in maintenance costs each year. Can be identified as a fixed increment or as a percent increase (similar to compound interest).

Maintenance Schedule

The maintenance schedule provides the user with an opportunity to identify particular maintenance treatments which are anticipated during a particular year of the roadway sections life. For example, the designer may expect that Hot Mix HL-4 Patching may be required in Year 7, at a cost of $1500/lane-km.

Add a Treatment

Add a maintenance treatment in a given year, i.e. Year 3. A treatment name and a cost must be included with each entry.

Delete Existing Treatment

Delete an existing, selected maintenance treatment.
Note: Do not leave the "Use Maintenance Schedule" table open without input.

Discount Rate

Compound rate used for calculating the present worth of future costs. Represents a blend between expected rate of return and expected rate of inflation.

4.0 Design Information and Analysis

The procedure for preparing a pavement design requires design data input and subsequent analysis. The format of the data input varies according to what type of design is selected. There are 2 basic types of design which are further divided into different material arrangements. These design types are shown below with their associated material arrangements.

New Pavement Design

Flexible (AC) Pavement

Rigid (PCC) Pavement

New Overlay Design

AC Overlay of AC Pavement

AC Overlay of PCC Pavement

AC Overlay of AC/PCC Pavement

Bonded PCC Overlay of PCC Pavement

Unbonded PCC Overlay of PCC Pavement

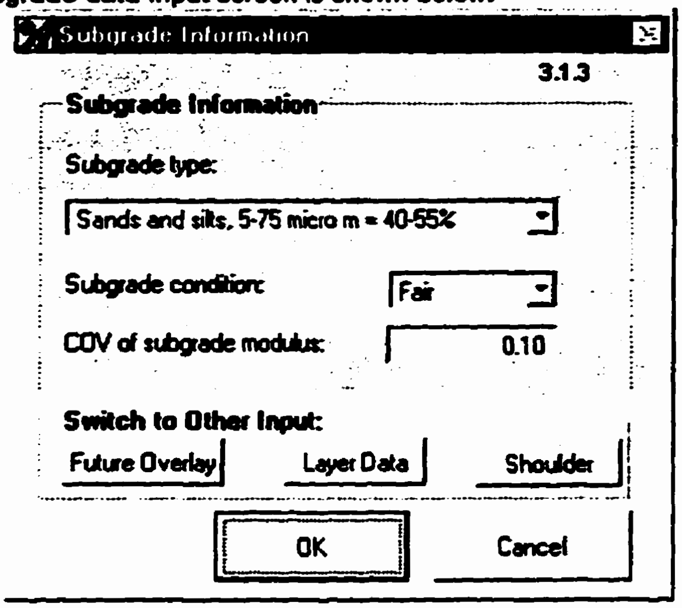
Once the data is input, the analysis can be performed by clicking of a mouse on the following line from the "Design" menu:

Design Alternatives

Flexible (AC) Pavement

Subgrade Data (AC)

The subgrade data input screen is shown below.



Subgrade Type

The subgrade type is used to determine the strength of the subgrade. The type may be chosen from the following picklist:

1. Gravels and sands,
2. Sands and Silts, 5-75μm<40%,
3. Sands and Silts, 5-75μm=40-55%,
4. Sands and Silts, 5-75μm>55%,
5. Lacustrine Clays,
6. Varved and Leda Clays.

Subgrade Condition

The subgrade condition is used to determine the strength of the subgrade. The type may be chosen from the following picklist:

1. Good
2. Fair
3. Poor

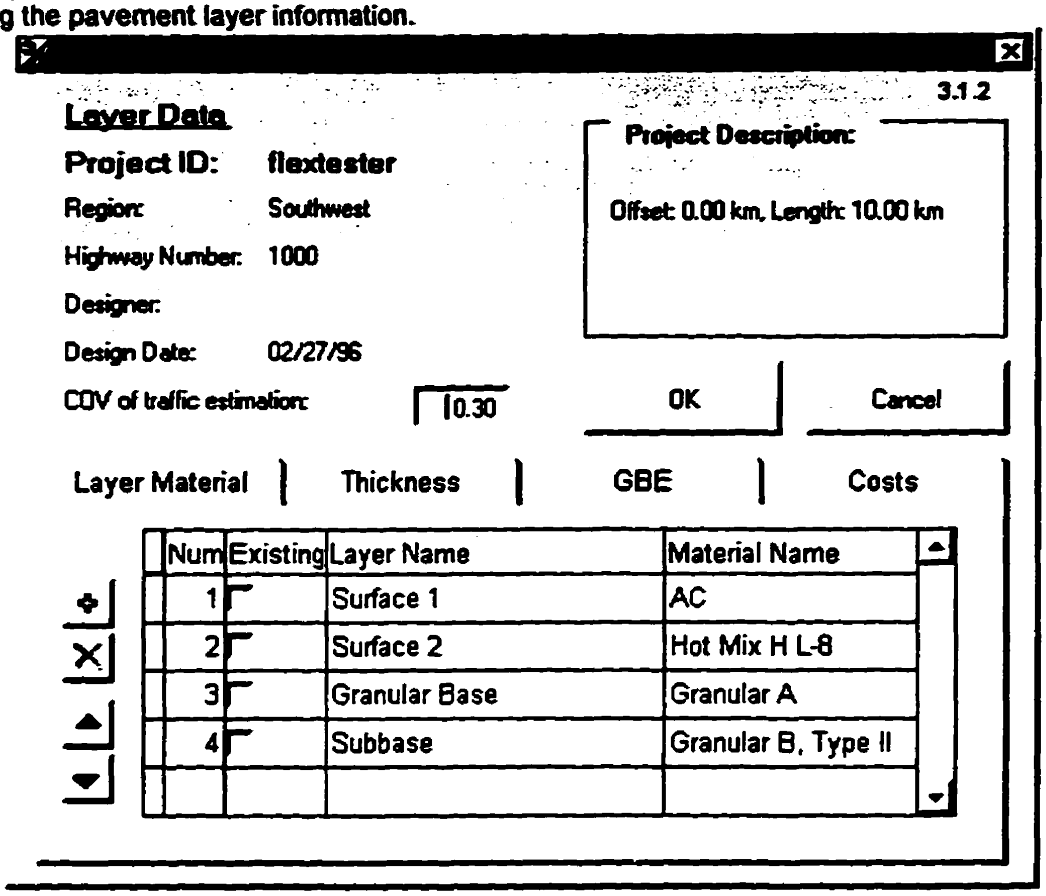
C.O.V. of Subgrade Strength

The Coefficient of Variance of subgrade strength is used to determine the uncertainty in subgrade strength. The coefficient of variance specifies the magnitude of the error in a design variable for reliability analysis.

Layer Data (AC)

Material

Material names are selected from the Material Table in the Data Library. If a material to be used in the design does not appear on the dropdown list, the Material Table needs to be edited prior to inputting the pavement layer information.



For overlay designs, the 'Existing' box should be checked for all layers that already exist.

C.O.V. of Traffic Estimation

The Coefficient of Variance of traffic estimation is used to determine the uncertainty in traffic estimation. The coefficient of variance specifies the magnitude of the error in a design variable for reliability analysis.

Thickness

Use this field to define the thickness range for each layer.

Layer Material | Thickness | GBE | Costs

Num	Existing	Lower (mm)	Upper (mm)	Inc (mm)
1		38	78	10
2		57	67	10
3		150	160	10
[illegible]		375	385	10

Lower Boundary of Layer Depth

Defines the lower limit of thickness for the design layer. This value must be entered in millimetres.

Upper Boundary of Layer Depth

Defines the upper limit of thickness for the design layer. This value must be entered in millimetres.

Increment of Layer Depth

Determines the number of design alternatives based on the upper and lower boundaries of layer depth. The smaller the increment, the more design alternatives - this should be considered during design, as it may affect computing time.

GBE

Granular base equivalence factors.

C.O.V. of GBE

Coefficient of Variance of the GBE is used to determine the uncertainty in GBE factors. The COV specifies the magnitude of the error in a design variable for reliability analysis.

Layer Material | Thickness | GBE | Costs

Num	Existing	GBE	COV of GBE	Drain.Factor	Material Name
1		2.00	0.10	1.00	H L-1
2		1.25	0.10	1.00	H L-4
3		1.00	0.10	1.00	GRAN A
4		0.67	0.10	1.00	GRAN B

Drainage Factor

Drainage factors are currently set to 1.0 for all materials. Further studies may suggest using a value smaller than 1.0 for individual materials.

Costs

Unit cost of materials. It can be in $/cu.m or $/Tonne.

Layer Material | Thickness | GBE | Costs

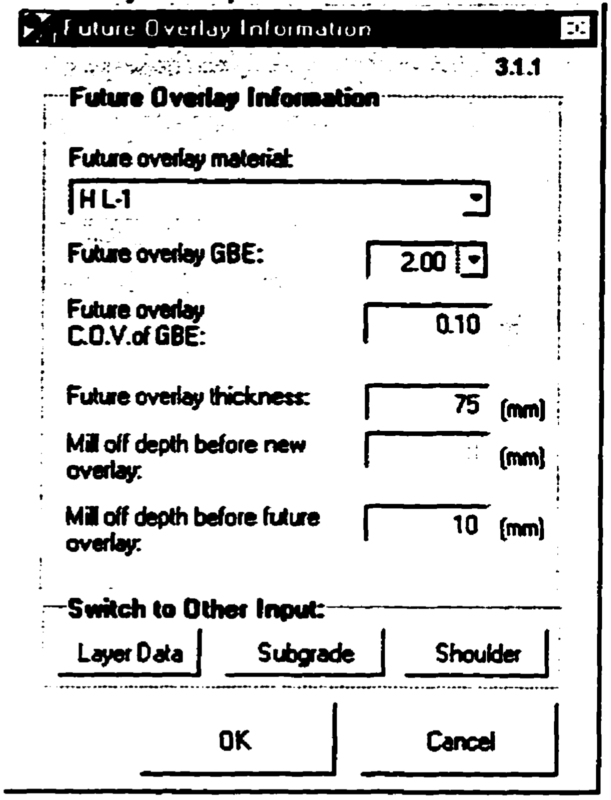
Num	Existing	Unit Cost	Units	Salvage Return (%)
1		50.00	cu.m	20
2		33.00	T	20
3		8.00	T	20
4		6.00	T	20

Dummy Layer

Allows using a layer that is only counted for the cost.

Future Overlay Data (AC)

The future overlay data input screen is shown below.

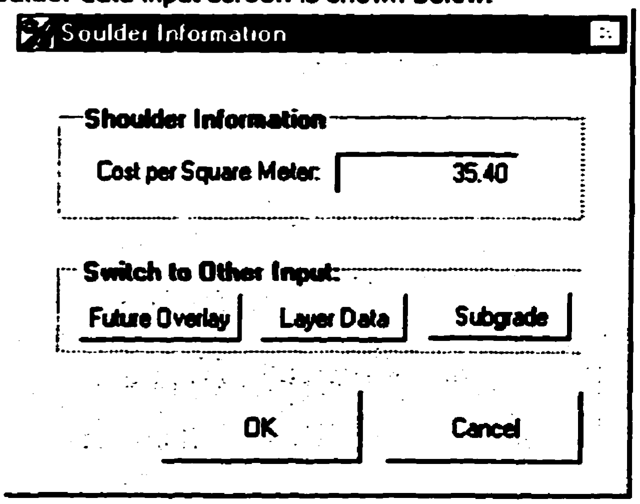


Future Overlay Depth (AC)

Defines the depth of future AC overlays which may be applied, if necessary, beyond the initial life of the pavement structure.

Shoulder Data (AC)

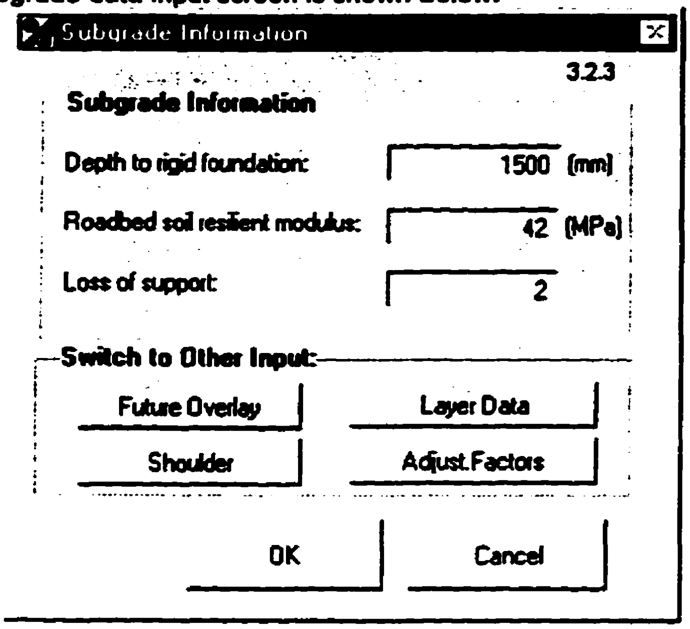
The shoulder data input screen is shown below.



Rigid (PCC) Pavement

Subgrade Data (PCC)

The subgrade data input screen is shown below.



Roadbed Soil Resilient Modulus (M_R)

AASHTO Guide suggests using a relationship of 1500*CBR for the subgrade resilient modulus, Mr (psi). Typical value: 7-345 MPa.

Loss of Support (LS)

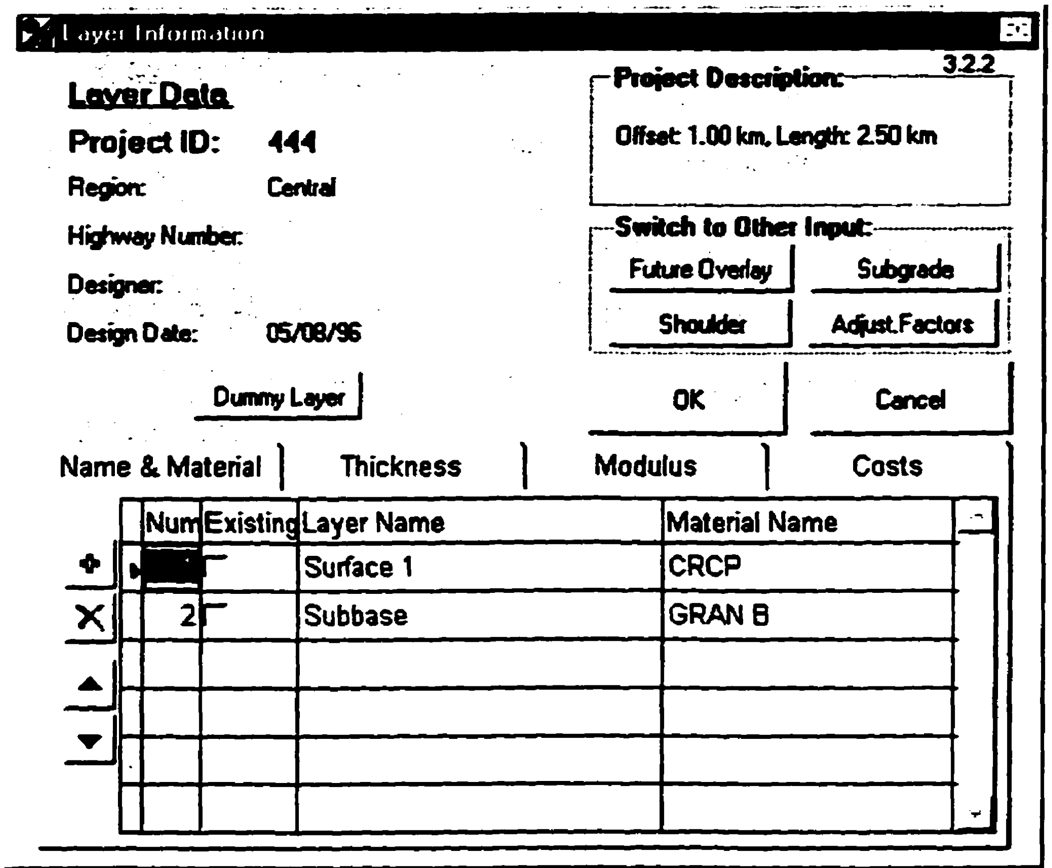
Typical LS values:

Type of material	LS
Cement treated granular base (E=7000-14000 MPa)	0 to 1
Cement aggregate mixtures (E=3450-7000 MPa)	0 to 1
Asphalt treated base (E=2400-7000 MPa)	0 to 1
Bituminous stabilized mixtures (E=280-2100 MPa)	0 to 1
Lime stabilized (E=140-480 MPa)	1 to 3
Unbound granular materials (E=100-310 MPa)	1 to 3
Fine grained or natural subgrade materials (E=20-280 MPa)	2 to 3

Layer Data (PCC)

Material

Material names are selected from the Material Table in the Data Library. If a material to be used in the design does not appear on the dropdown list, the Material Table needs to be edited prior to inputting the pavement layer information.



Thickness

Use this field to define the thickness range for each layer.

Name & Material | Thickness | Modulus | Costs

Num	Existing	Lower (mm)	Upper (mm)	Inc (mm)
1		100	250	50
2		200	300	50

Lower Boundary of Layer Depth

Defines the lower limit of thickness for the design layer. This value must be entered in millimetres.

Upper Boundary of Layer Depth

Defines the upper limit of thickness for the design layer. This value must be entered in millimetres.

Increment of Layer Depth

Determines the number of design alternatives based on the upper and lower boundaries of layer depth. The smaller the increment, the more design alternatives - this should be considered during design, as it may affect computing time.

Modulus

Use this field to define material moduli.

Name & Material | Thickness | **Modulus** | Costs

Num	Existing	Rupture	Elasticity	Load Transf	Material Name
1		4.00	29000	2.80	CRCP
2			140		GRAN B

Modulus of Rupture

AASHTO suggests the 28-day mean value Sc' be calculated as:

$$Sc'(\text{mean}) = Sc + z(SD_s)$$

where,

Sc is construction specification on concrete modulus of rupture (MPa)
Sd_s = estimated standard deviation of concrete modulus of rupture (MPa)
z = standard normal deviate

Please consult the Engineering Document for more details.

Modulus of Elasticity (Ec)

The modulus of elasticity (Ec) of the concrete can be approximated as:

$$Ec = 6.750 \cdot Sc' \text{ (MPa)}$$

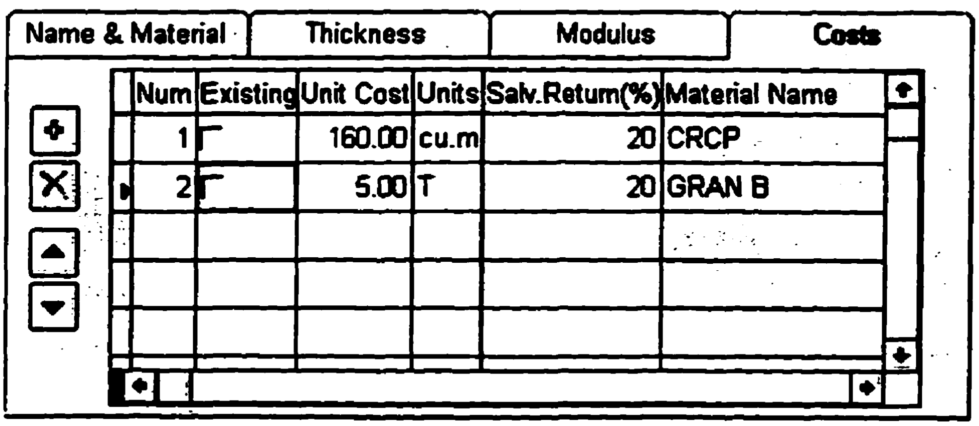
Load Transfer Coefficient (J)

Recommended Load Transfer Coefficient

Shoulder	Asphalt		Tied PCC	
Load Transfer Devices	Yes	No	Yes	No
Pavement Type				
Plain jointed and jointed reinforced	3.2	3.8-4.4	2.5-3.1	3.6-4.2
CRCP	2.9-3.2	N/A	2.3-2.9	N/A

Costs

Unit cost of materials. It can be in $/cu.m or $/Tonne.



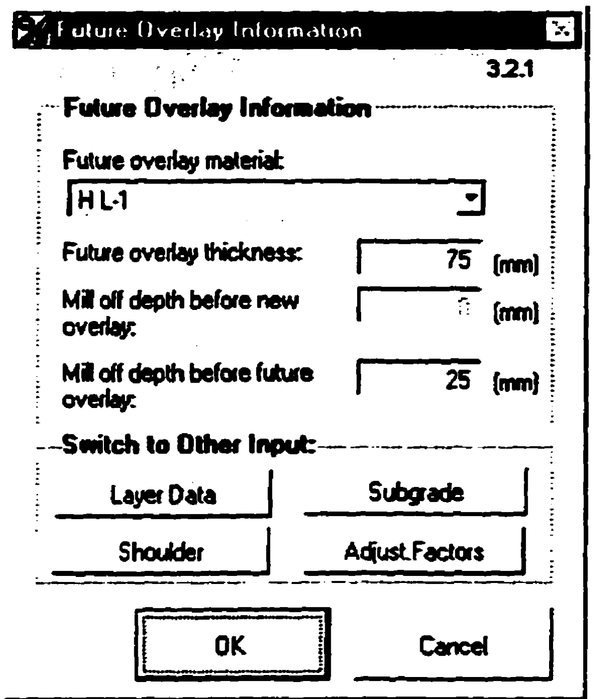
Num	Existing	Unit Cost	Units	Salv.Return(%)	Material Name
1		160.00	cu.m	20	CRCP
2		5.00	T	20	GRAN B

Dummy Layer

Allows using a layer that is only counted for the cost.

Future Overlay Data

The future overlay data input screen is shown below.

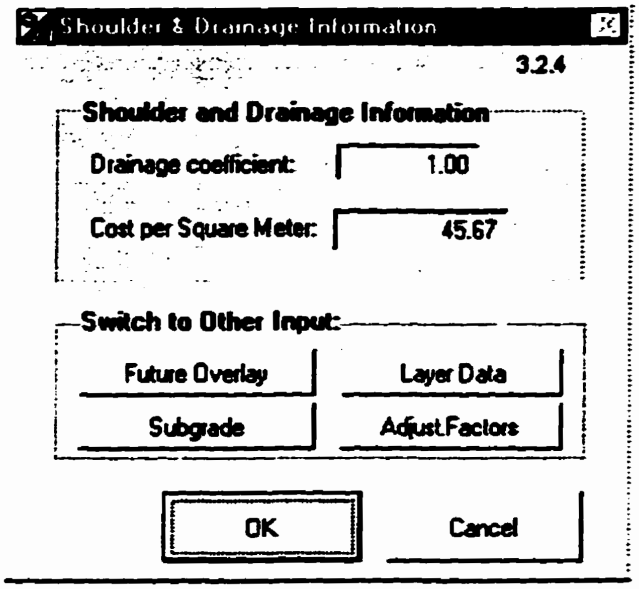


Future Overlay Thickness

Defines the depth of future overlays which may be applied, if necessary, beyond the initial life of the pavement structure.

Shoulder and Drainage Data

The shoulder and drainage data input screen is shown below.



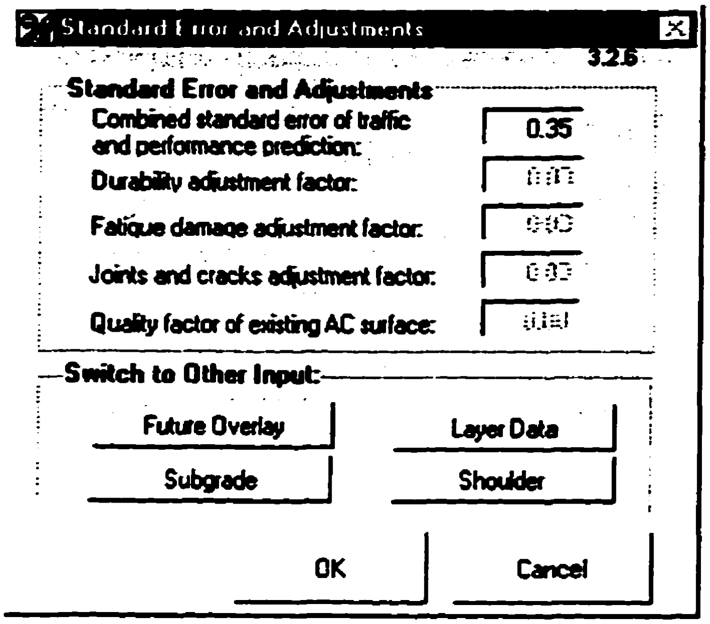
Drainage Coefficient (Cd)

Recommended Drainage Coefficient, C_d

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	<1%	1-5%	5-25%	>25%
Excellent	1.25-1.20	1.20-1.15	1.15-1.10	1.10
Good	1.20-1.15	1.15-1.10	1.10-1.00	1.00
Fair	1.15-1.10	1.10-1.00	1.00-0.90	0.90
Poor	1.10-1.00	1.00-0.90	0.90-0.80	0.80
Very Poor	1.00-0.90	0.90-0.80	0.80-0.70	0.70

Standard Error and Adjustment Factors (PCC)

The Standard Error and Adjustment Factors input screen is shown below. Adjustment Factors are used in new overlay designs.



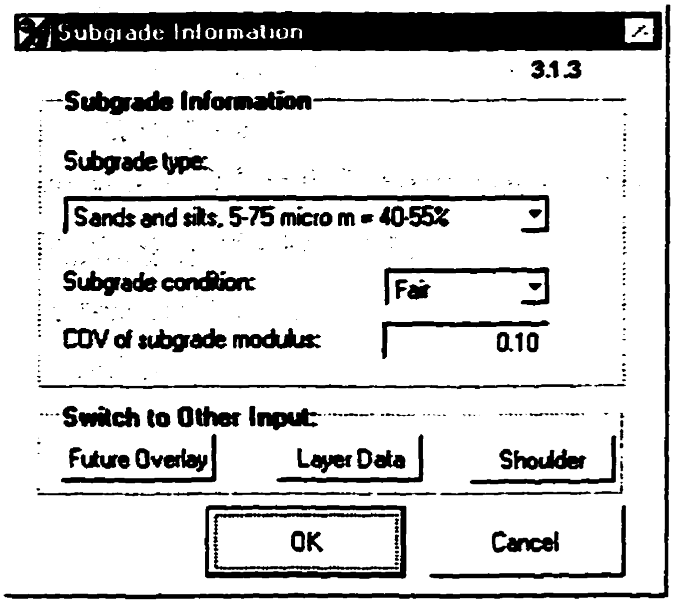
Combined Standard Error (S_0)

The combined standard error of the traffic and performance prediction is a coefficient used in deriving the AASHTO design equations. A range of 0.3 - 0.4 is recommended by AASHTO for rigid pavements.

AC Overlay of AC Pavement

Subgrade Data (AC)

The subgrade data input screen is shown below.



Subgrade Type

The subgrade type is used to determine the strength of the subgrade. The type may be chosen from the following picklist:

1. Gravels and sands,
2. Sands and Silts, 5-75μm<40%,
3. Sands and Silts, 5-75μm=40-55%,
4. Sands and Silts, 5-75μm>55%,
5. Lacustrine Clays,
6. Varved and Leda Clays.

Subgrade Condition

The subgrade condition is used to determine the strength of the subgrade. The type may be chosen from the following picklist:

1. Good
2. Fair
3. Poor

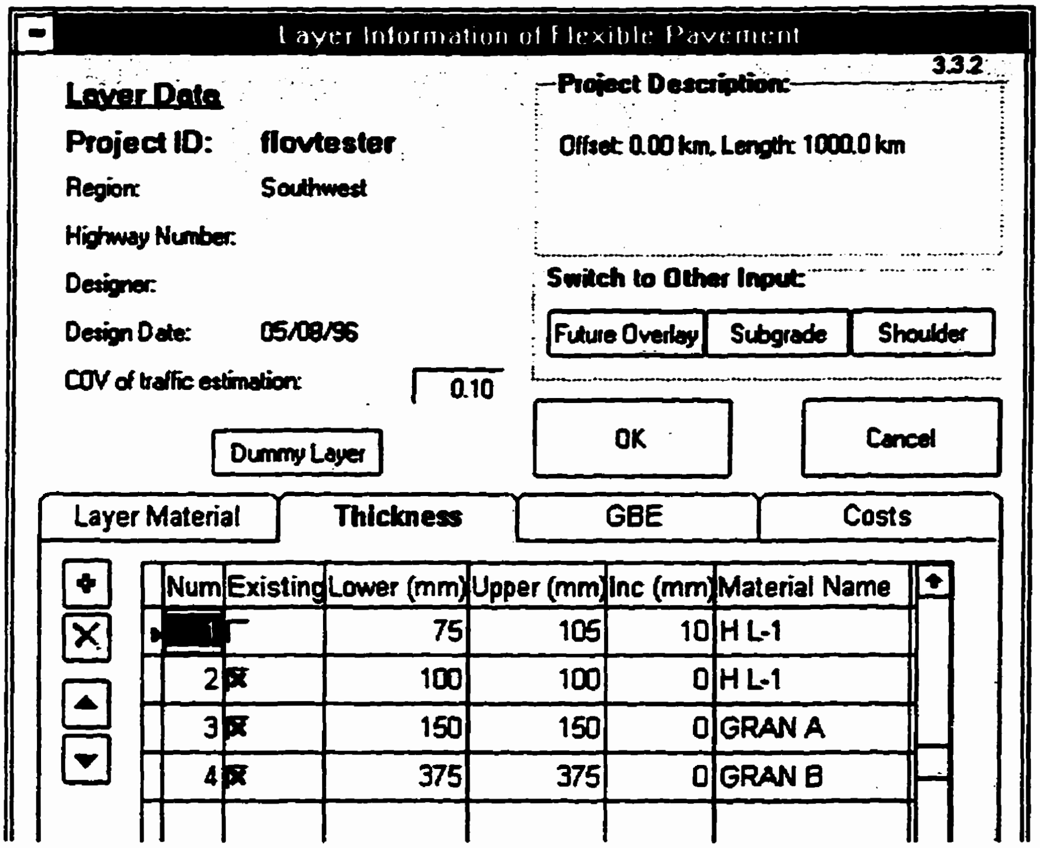
C.O.V. of Subgrade Strength

The Coefficient of Variance of subgrade strength is used to determine the uncertainty in subgrade strength. The coefficient of variance specifies the magnitude of the error in a design variable for reliability analysis.

Layer Data (AC)

Material

Material names are selected from the Material Table in the Data Library. If a material to be used in the design does not appear on the dropdown list, the Material Table needs to be edited prior to inputting the pavement layer information.



For overlay designs, the 'Existing' box should be checked for all layers that already exist.

C.O.V. of Traffic Estimation

The Coefficient of Variance of traffic estimation is used to determine the uncertainty in traffic estimation. The coefficient of variance specifies the magnitude of the error in a design variable for reliability analysis.

Thickness

Layer Material | Thickness | GBE | Costs

Num	Existing	Lower (mm)	Upper (mm)	Inc (mm)
1	☐	38	78	10
2	☐	57	67	10
3	☐	150	160	10
[illegible]	☐	375	385	10

Lower Boundary of Layer Depth

Defines the lower limit of thickness for the design layer. This value must be entered in millimetres.

Upper Boundary of Layer Depth

Defines the upper limit of thickness for the design layer. This value must be entered in millimetres.

Increment of Layer Depth

Determines the number of design alternatives based on the upper and lower boundaries of layer depth. The smaller the increment, the more design alternatives - this should be considered during design, as it may affect computing time.

GBE

Granular base equivalence factors.

C.O.V. of GBE

Coefficient of Variance of the GBE is used to determine the uncertainty in GBE factors. The COV specifies the magnitude of the error in a design variable for reliability analysis.

Layer Material | Thickness | GBE | Costs

Num	Existing	GBE	COV of GBE	Drain.Factor	Material Name
1	☐	2.00	0.10	1.00	H L-1
2	☒	2.00	0.10	1.00	H L-1
3	☒	1.00	0.10	1.00	GRAN A
4	☒	0.50	0.10	1.00	GRAN B

Drainage Factor

Drainage factors are currently set to 1.0 for all materials. Further studies may suggest using a value smaller than 1.0 for individual materials.

Costs

Unit cost of materials. It can be in $/cu.m or $/Tonne.

Layer Material | Thickness | GBE | Costs

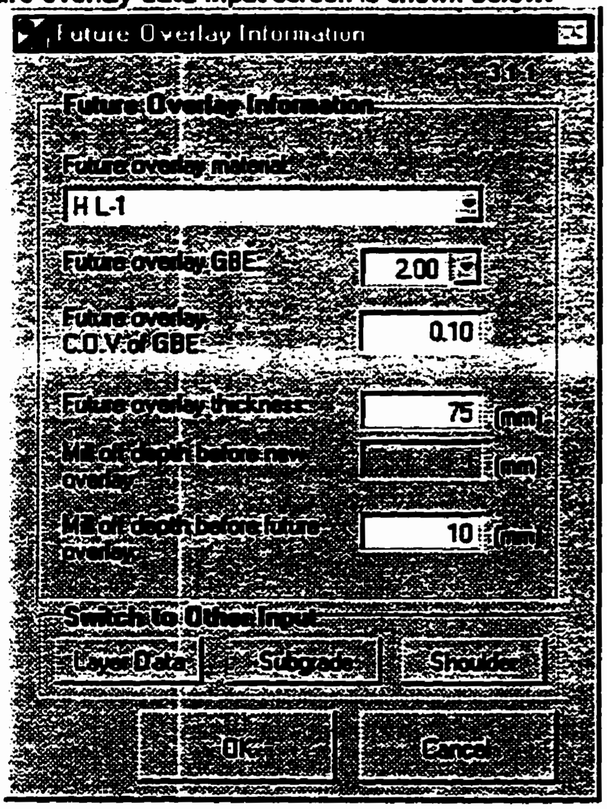
Num	Existing	Unit Cost	Units	Salv.Return(%)	Material Name
1		50.00	T	25	H L-1
2		0.00	T	25	H L-1
3		0.00	T	25	GRAN A
4		0.00	T	25	GRAN B

Dummy Layer

Allows using a layer that is only counted for the cost.

Future Overlay Data (AC)

The future overlay data input screen is shown below.

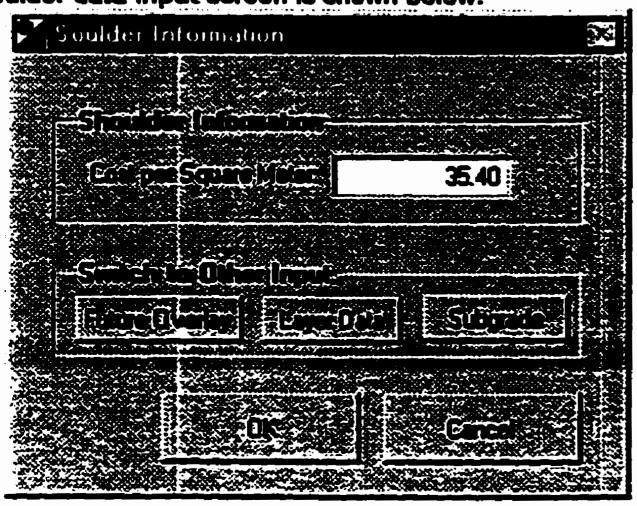


Future Overlay Depth (AC)

Defines the depth of future AC overlays which may be applied, if necessary, beyond the initial life of the pavement structure.

Shoulder Data (AC)

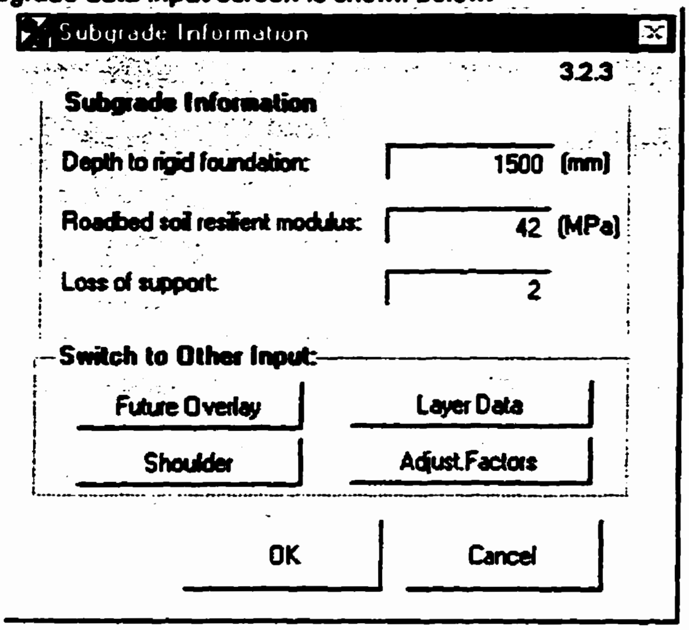
The shoulder data input screen is shown below.



AC Overlay of PCC Pavement

Subgrade Data (PCC)

The subgrade data input screen is shown below.



Roadbed Soil Resilient Modulus (M_R)

AASHTO Guide suggests using a relationship of 1500*CBR for the subgrade resilient modulus, Mr (psi). Typical value: 7-345 MPa.

Loss of Support (LS)

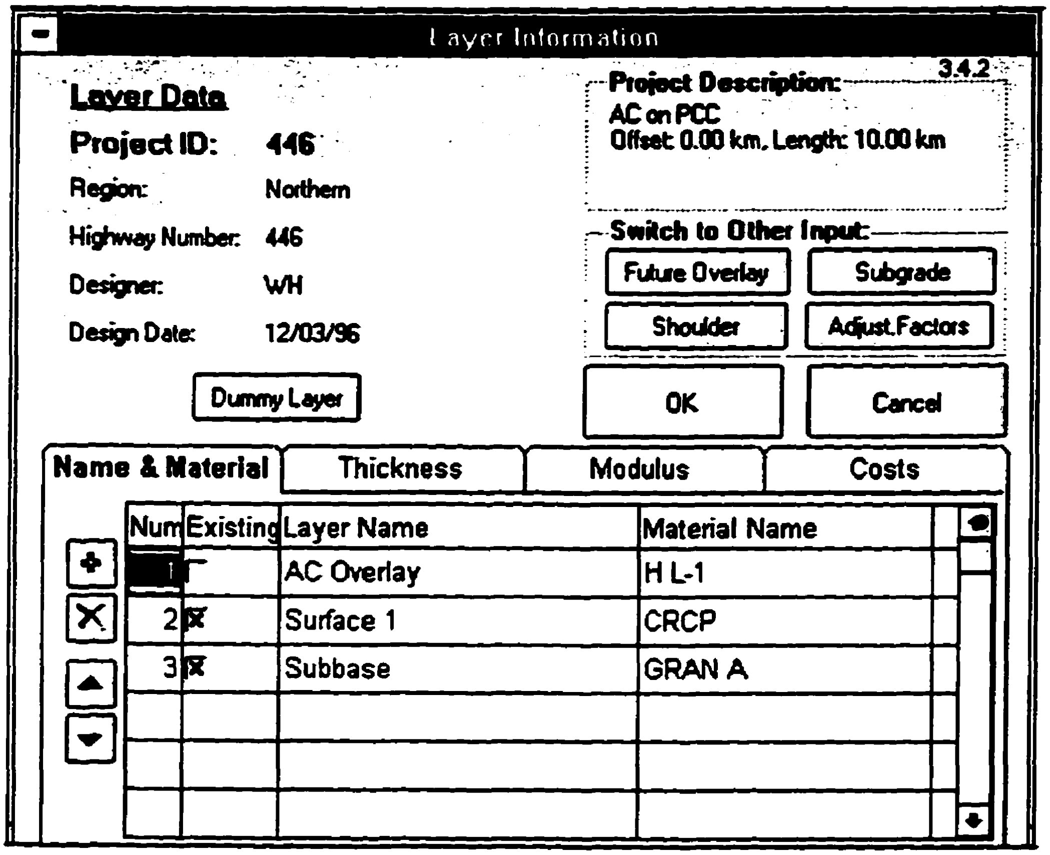
Typical LS values:

Type of material	LS
Cement treated granular base (E=7000-14000 MPa)	0 to 1
Cement aggregate mixtures (E=3450-7000 MPa)	0 to 1
Asphalt treated base (E=2400-7000 MPa)	0 to 1
Bituminous stabilized mixtures (E=280-2100 MPa)	0 to 1
Lime stabilized (E=140-480 MPa)	1 to 3
Unbound granular materials (E=100-310 MPa)	1 to 3
Fine grained or natural subgrade materials (E=20-280 MPa)	2 to 3

Layer Data (PCC)

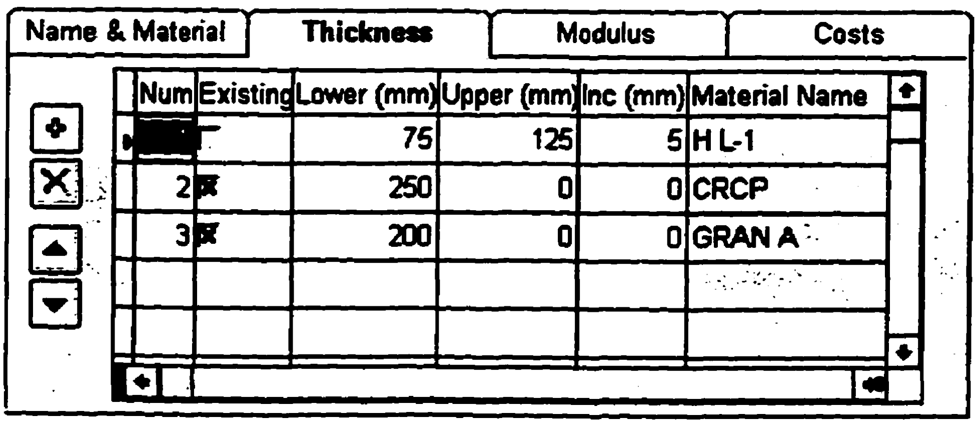
Material

Material names are selected from the Material Table in the Data Library. If a material to be used in the design does not appear on the dropdown list, the Material Table needs to be edited prior to inputting the pavement layer information.



Thickness

Use this field to define the thickness range for each layer.



Lower Boundary of Layer Depth

Defines the lower limit of thickness for the design layer. This value must be entered in millimetres.

Upper Boundary of Layer Depth

Defines the upper limit of thickness for the design layer. This value must be entered in millimetres.

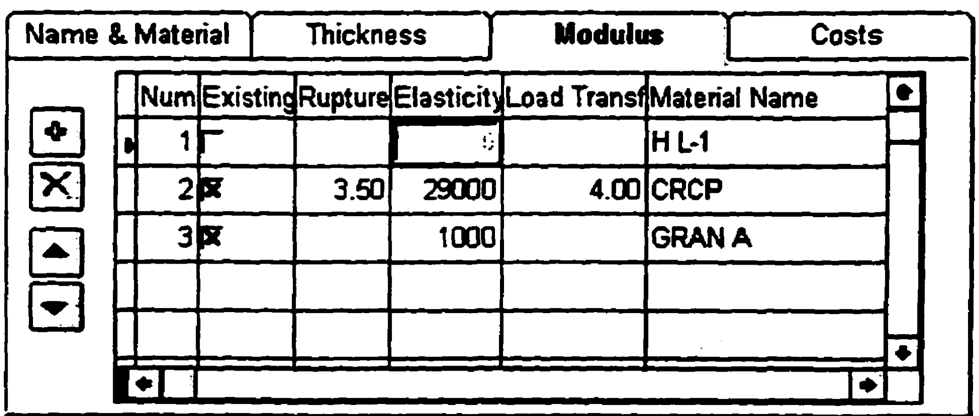
Increment of Layer Depth

Determines the number of design alternatives based on the upper and lower boundaries of layer depth. The smaller the increment, the more design alternatives - this should be considered during design, as it may affect computing time.

Thickness of Existing PCC Slab (D)

The thickness of the existing PCC slab should be obtained through review of original design and construction documents. Coring of the existing pavement is recommended to ensure the accuracy of this input.

Modulus



Modulus of Rupture

AASHTO suggests the 28-day mean value Sc' be calculated as:

$Sc'(mean) = Sc + z(SD_s)$

where,

Sc is construction specification on concrete modulus of rupture (MPa)
SDs = estimated standard deviation of concrete modulus of rupture (MPa)
z = standard normal deviate

Please consult the Engineering Document for more details.

Modulus of Elasticity (Ec)

The modulus of elasticity (Ec) of the concrete can be approximated as:

$Ec = 6.750 \cdot Sc'$ (MPa)

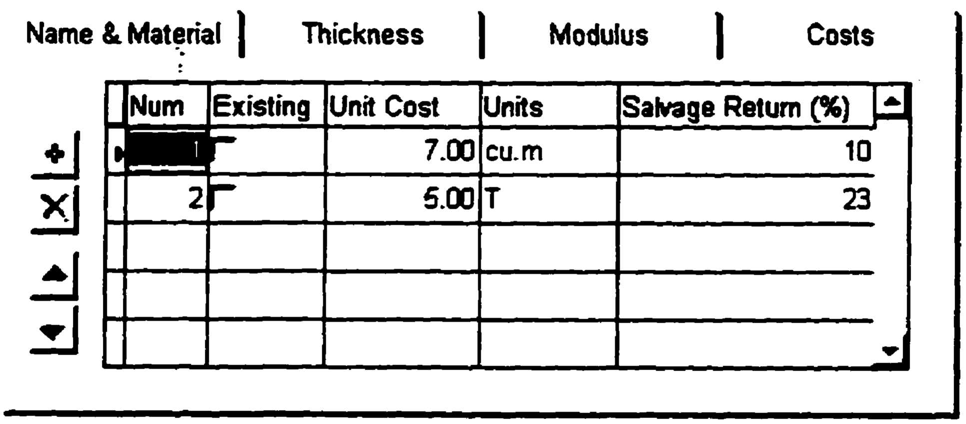
Load Transfer Coefficient (J)

Recommended Load Transfer Coefficient

Shoulder	**Asphalt**		**Tied PCC**	
Load Transfer Devices	**Yes**	**No**	**Yes**	**No**
Pavement Type				
Plain jointed and jointed reinforced	3.2	3.8-4.4	2.5-3.1	3.6-4.2
CRCP	2.9-3.2	N/A	2.3-2.9	N/A

Costs

Unit cost of materials. It can be in $/cu.m or $/Tonne.

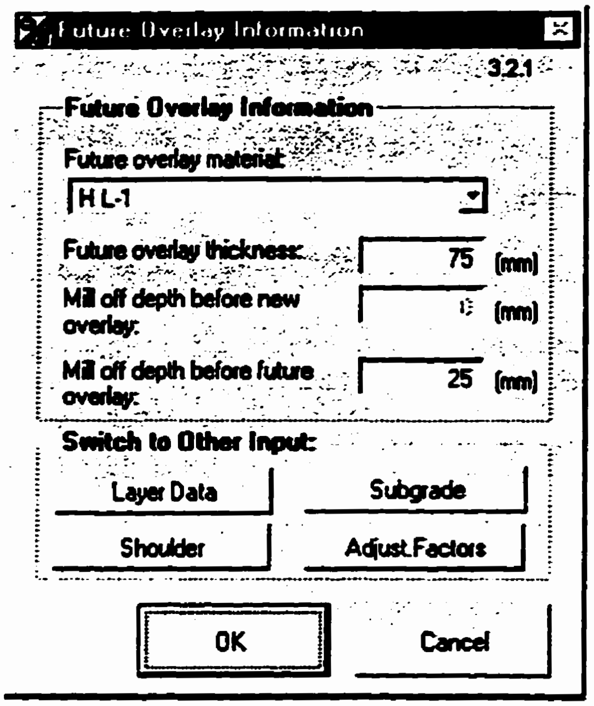


Dummy Layer

Allows using a layer that is only counted for the cost.

Future Overlay Data

The future overlay data input screen is shown below.

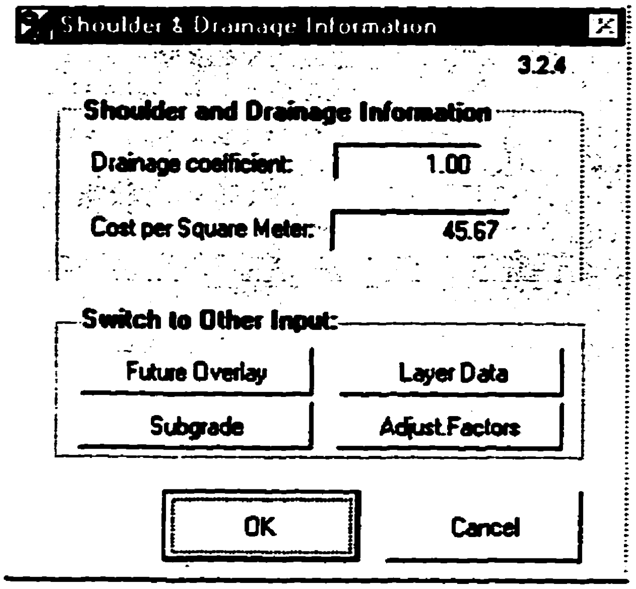


Future Overlay Thickness

Defines the depth of future overlays which may be applied, if necessary, beyond the initial life of the pavement structure.

Shoulder and Drainage Data

The shoulder and drainage data input screen is shown below.



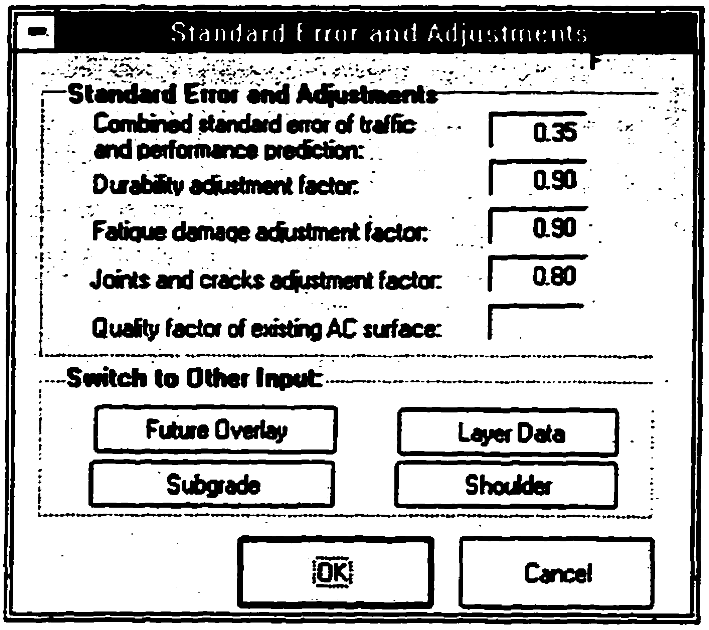
Drainage Coefficient (Cd)

Recommended Drainage Coefficient, C_d

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	<1%	1-5%	5-25%	>25%
Excellent	1.25-1.20	1.20-1.15	1.15-1.10	1.10
Good	1.20-1.15	1.15-1.10	1.10-1.00	1.00
Fair	1.15-1.10	1.10-1.00	1.00-0.90	0.90
Poor	1.10-1.00	1.00-0.90	0.90-0.80	0.80
Very Poor	1.00-0.90	0.90-0.80	0.80-0.70	0.70

Standard Error and Adjustment Factors (PCC)

The Standard Error and Adjustment Factors input screen is shown below.



Combined Standard Error (S_0)

The combined standard error of the traffic and performance prediction is a coefficient used in deriving the AASHTO design equations. A range of 0.3 - 0.4 is recommended by AASHTO for rigid pavements.

Joints and Cracks Adjustment Factor (Fjc)

According to Figure 5.12 in the AASHTO guide, this factor can be determined based on the condition survey of the existing PCC pavement. Recommended value 1.0, repair all deteriorated areas.

- 1.0-0.84: 0-40 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km
- 0.84-0.70: 41-80 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km
- 0.70-0.56: 81-120 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km
- 0.56: > 120 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km

Durability Adjustment Factor (Fdur)

This factor is determined based on the condition survey of the existing PCC pavement:

- 1.00: No sign of PCC durability problems
- 0.96-0.99: Some durability cracking exist, but no spalling
- 0.88-0.95: Substantial cracking and some spalling exist
- 0.80-0.88: Extensive cracking and severe spalling exist

Fatigue Damage Adjustment Factor (Ffat)

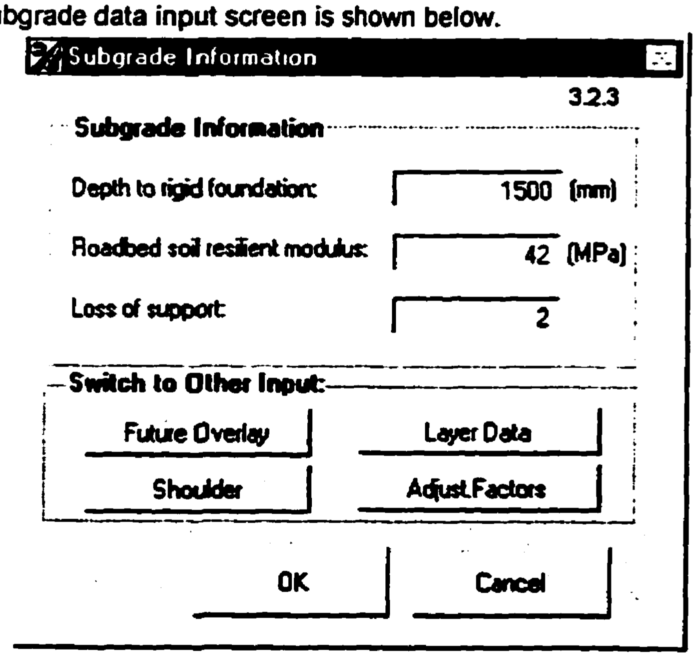
This factor is determined based on the condition survey of the existing PCC pavement:

- 0.97-1.00: Few transverse cracks/punchouts exist
  JPCP: < 5 percent slabs are cracked
  JRCP: < 16 working cracks per km
  CRCP: < 3 punchouts per km
- 0.94-0.96: A significant number of transverse cracks/punchouts exist
  JPCP: 5-15 percent slabs are cracked
  JRCP: 16-47 working cracks per km
  CRCP: 3-7 punchouts per km
- 0.90-0.93: A large number of transverse cracks/punchouts exist
  JPCP: > 15 percent slabs are cracked
  JRCP: > 47 working cracks per km
  CRCP: > 7 punchouts per km

AC Overlay of AC/PCC Pavement

Subgrade Data (PCC)

The subgrade data input screen is shown below.



Roadbed Soil Resilient Modulus (M_R)

AASHTO Guide suggests using a relationship of 1500*CBR for the subgrade resilient modulus, Mr (psi). Typical value: 7-345 MPa.

Loss of Support (LS)

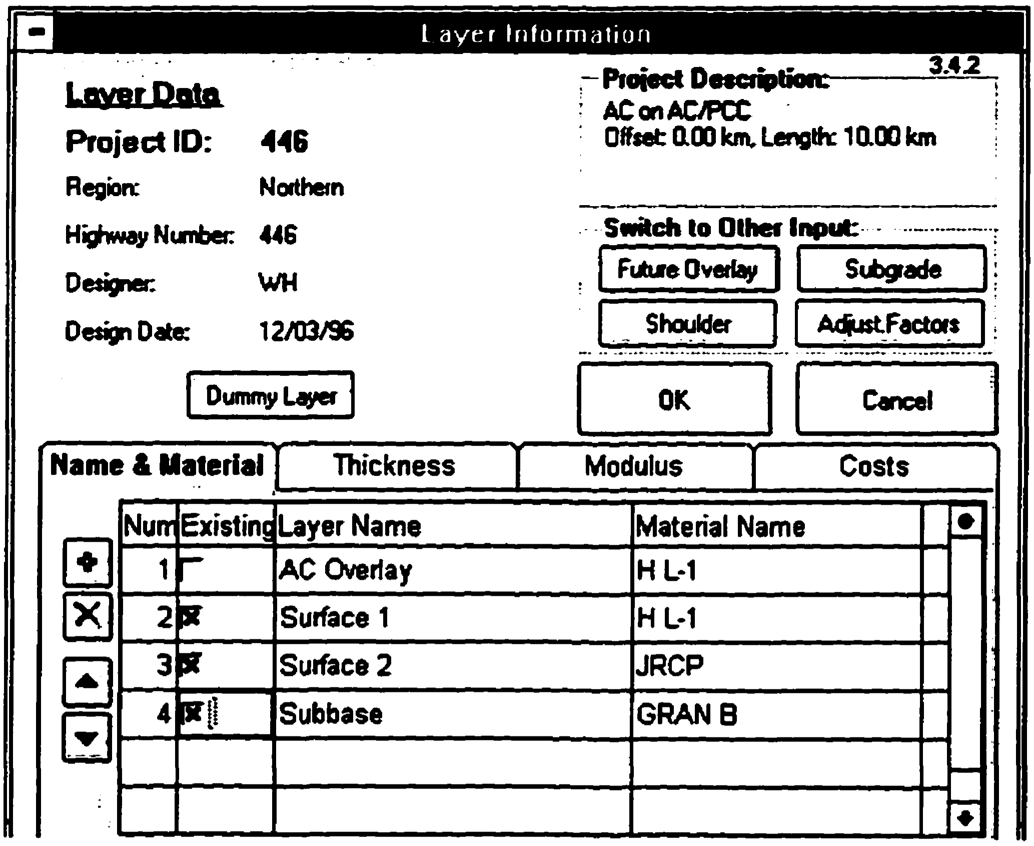
Typical LS values:

Type of material	LS
Cement treated granular base (E=7000-14000 MPa)	0 to 1
Cement aggregate mixtures (E=3450-7000 MPa)	0 to 1
Asphalt treated base (E=2400-7000 MPa)	0 to 1
Bituminous stabilized mixtures (E=280-2100 MPa)	0 to 1
Lime stabilized (E=140-480 MPa)	1 to 3
Unbound granular materials (E=100-310 MPa)	1 to 3
Fine grained or natural subgrade materials (E=20-280 MPa)	2 to 3

Layer Data (PCC)

Material

Material names are selected from the Material Table in the Data Library. If a material to be used in the design does not appear on the dropdown list, the Material Table needs to be edited prior to inputting the pavement layer information.



Thickness

Use this field to define the thickness range for each layer.

Name & Material | Thickness | Modulus | Costs

Num	Existing	Lower (mm)	Upper (mm)	Inc (mm)	Material Name
1		75	125	5	H L-1
2	X	150	0	0	H L-1
3	X	200	0	0	JRCP
4	X	250	0	0	GRAN B

Lower Boundary of Layer Depth

Defines the lower limit of thickness for the design layer. This value must be entered in millimetres.

Upper Boundary of Layer Depth

Defines the upper limit of thickness for the design layer. This value must be entered in millimetres.

Increment of Layer Depth

Determines the number of design alternatives based on the upper and lower boundaries of layer depth. The smaller the increment, the more design alternatives - this should be considered during design, as it may affect computing time.

Thickness of Existing PCC Slab (D)

The thickness (in mm) of the existing PCC slab should be obtained through review of original design from construction documents. Coring of the existing pavement is recommended to ensure the accuracy of this input.

Modulus

Name & Material | Thickness | Modulus | Costs

Num	Existing	Rupture	Elasticity	Load Transf	Material Name
1					H L-1
2	X	0.00	0	0.00	H L-1
3	X	3.60	40000	0.00	JRCP
4	X	0.00	140	0.00	GRAN B

Modulus of Rupture

AASHTO suggests the 28-day mean value Sc' be calculated as:

$$Sc'(mean) = Sc + z(SD_s)$$

where,

Sc is construction specification on concrete modulus of rupture (MPa)

SDs = estimated standard deviation of concrete modulus of rupture (MPa)

z = standard normal deviate

Please consult the Engineering Document for more details.

Modulus of Elasticity (Ec)

The modulus of elasticity (Ec) of the concrete can be approximated as:

$$Ec = 6.750 * Sc' \text{ (MPa)}$$

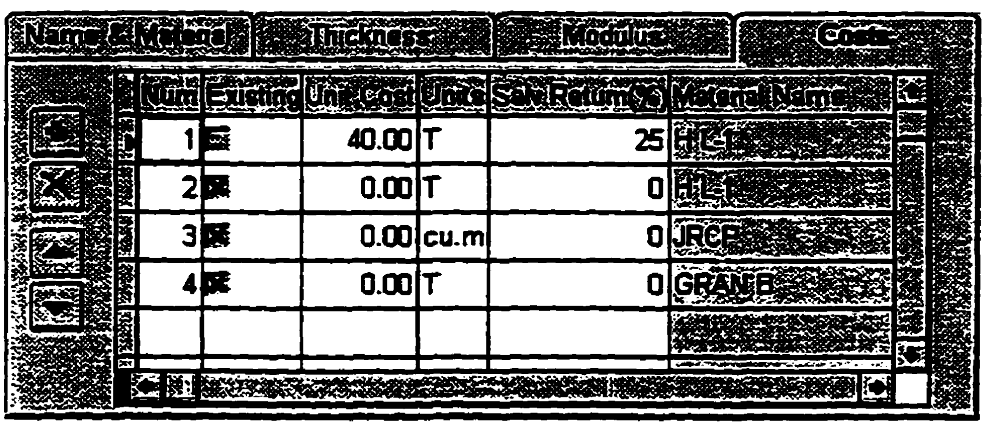
Load Transfer Coefficient (J)

Recommended Load Transfer Coefficient

Shoulder	Asphalt		Tied PCC	
Load Transfer Devices	Yes	No	Yes	No
Pavement Type				
Plain jointed and jointed reinforced	3.2	3.8-4.4	2.5-3.1	3.6-4.2
CRCP	2.9-3.2	N/A	2.3-2.9	N/A

Costs

Unit cost of materials. It can be in $/cu.m or $/Tonne.

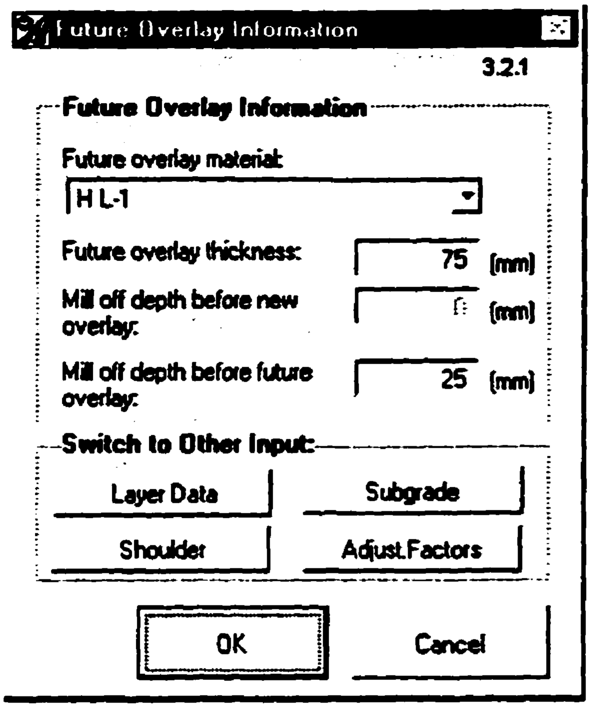


Dummy Layer

Allows using a layer that is only counted for the cost.

Future Overlay Data

The future overlay data input screen is shown below.

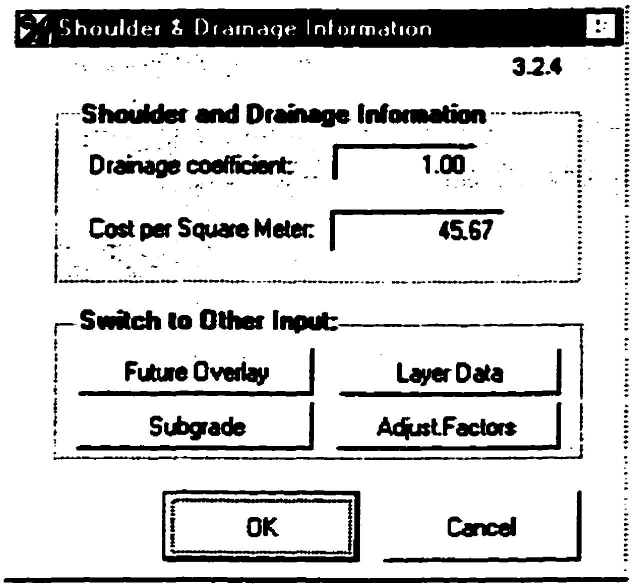


Future Overlay Thickness (PCC)

Defines the depth of future overlays which may be applied, if necessary, beyond the initial life of the pavement structure.

Shoulder and Drainage Data

The shoulder and drainage data input screen is shown below.



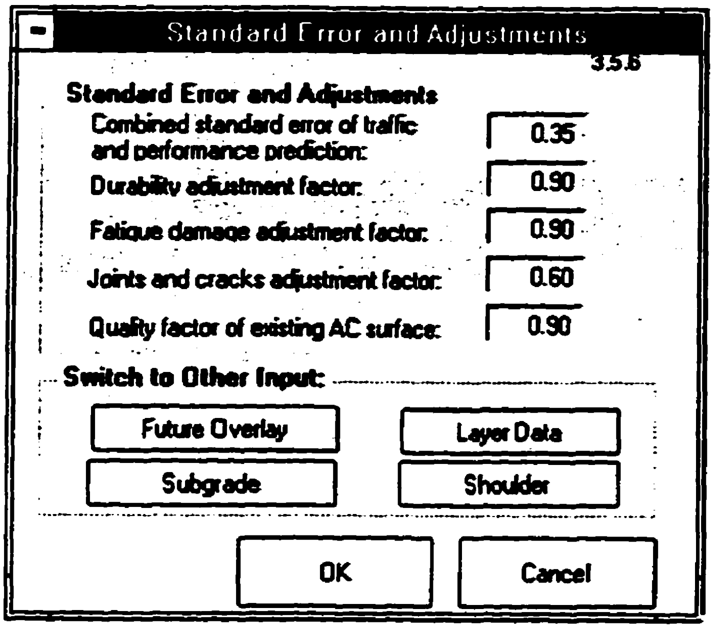
Drainage Coefficient (Cd)

Recommended Drainage Coefficient, C_d

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	<1%	1-5%	5-25%	>25%
Excellent	1.25-1.20	1.20-1.15	1.15-1.10	1.10
Good	1.20-1.15	1.15-1.10	1.10-1.00	1.00
Fair	1.15-1.10	1.10-1.00	1.00-0.90	0.90
Poor	1.10-1.00	1.00-0.90	0.90-0.80	0.80
Very Poor	1.00-0.90	0.90-0.80	0.80-0.70	0.70

<u>Standard Error and Adjustment Factors (PCC)</u>

The Standard Error and Adjustment Factors input screen is shown below.



Combined Standard Error (S_0)

The combined standard error of the traffic and performance prediction is a coefficient used in deriving the AASHTO design equations. A range of 0.3 - 0.4 is recommended by AASHTO for rigid pavements.

Joints and Cracks Adjustment Factor (Fjc)

According to Figure 5.12 in the AASHTO guide, this factor can be determined based on the condition survey of the existing PCC pavement. Recommended value 1.0, repair all deteriorated areas.

- 1.0-0.84: 0-40 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km
- 0.84-0.70: 41-80 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km
- 0.70-0.56: 81-120 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km
- 0.56: > 120 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km

Durability Adjustment Factor (Fdur)

This factor is determined based on the condition survey of the existing PCC pavement:

- 1.00: No sign of PCC durability problems
- 0.96-0.99: Some durability cracking exist, but no spalling
- 0.88-0.95: Substantial cracking and some spalling exist
- 0.80-0.88: Extensive cracking and severe spalling exist

Fatigue Damage Adjustment Factor (Ffat)

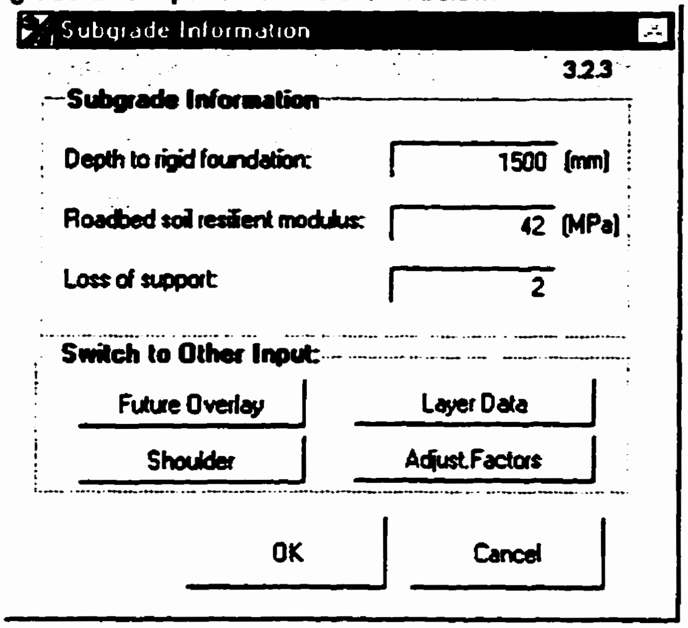
This factor is determined based on the condition survey of the existing PCC pavement:

- 0.97-1.00: Few transverse cracks/punchouts exist
  JPCP: < 5 percent slabs are cracked
  JRCP: < 16 working cracks per km
  CRCP: < 3 punchouts per km
- 0.94-0.96: A significant number of transverse cracks/punchouts exist
  JPCP: 5-15 percent slabs are cracked
  JRCP: 16-47 working cracks per km
  CRCP: 3-7 punchouts per km
- 0.90-0.93: A large number of transverse cracks/punchouts exist
  JPCP: > 15 percent slabs are cracked
  JRCP: > 47 working cracks per km
  CRCP: > 7 punchouts per km

Bonded PCC Overlay of PCC Pavement

Subgrade Data (PCC)

The subgrade data input screen is shown below.



Roadbed Soil Resilient Modulus (M_R)

AASHTO Guide suggests using a relationship of 1500*CBR for the subgrade resilient modulus, Mr (psi). Typical value: 7-345 MPa.

Loss of Support (LS)

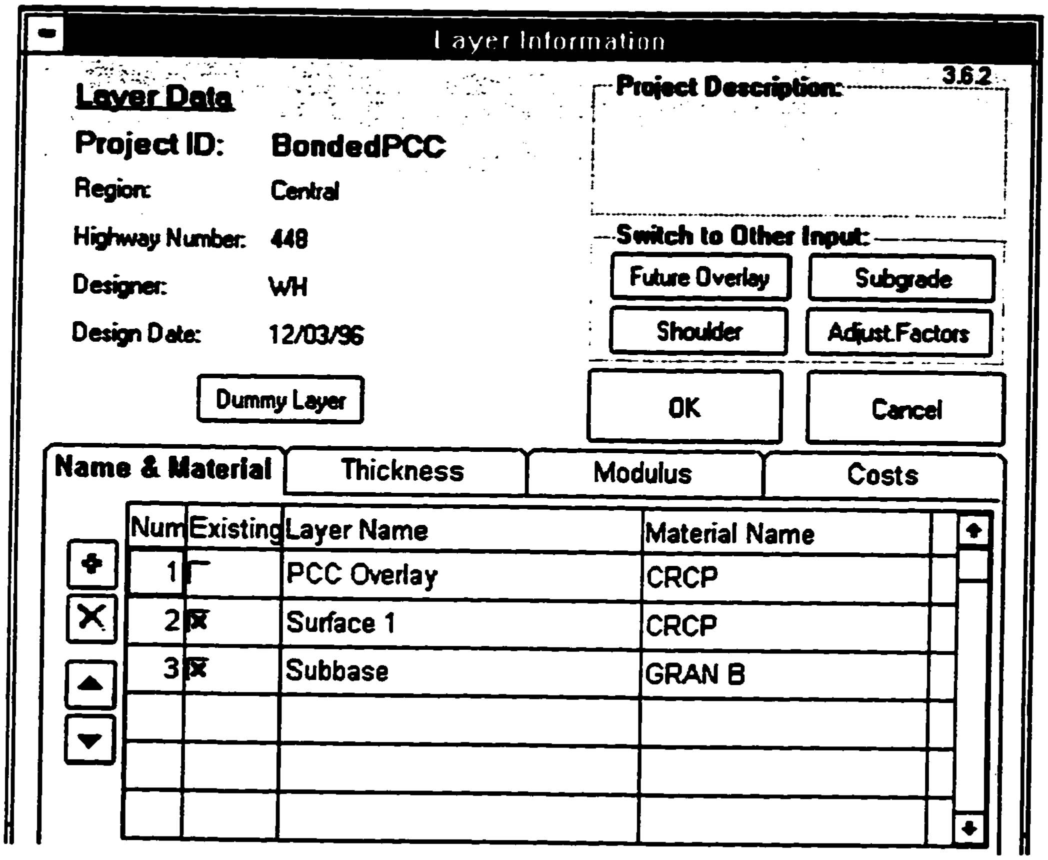
Typical LS values:

Type of material	LS
Cement treated granular base (E=7000-14000 MPa)	0 to 1
Cement aggregate mixtures (E=3450-7000 MPa)	0 to 1
Asphalt treated base (E=2400-7000 MPa)	0 to 1
Bituminous stabilized mixtures (E=280-2100 MPa)	0 to 1
Lime stabilized (E=140-480 MPa)	1 to 3
Unbound granular materials (E=100-310 MPa)	1 to 3
Fine grained or natural subgrade materials (E=20-280 MPa)	2 to 3

Layer Data (PCC)

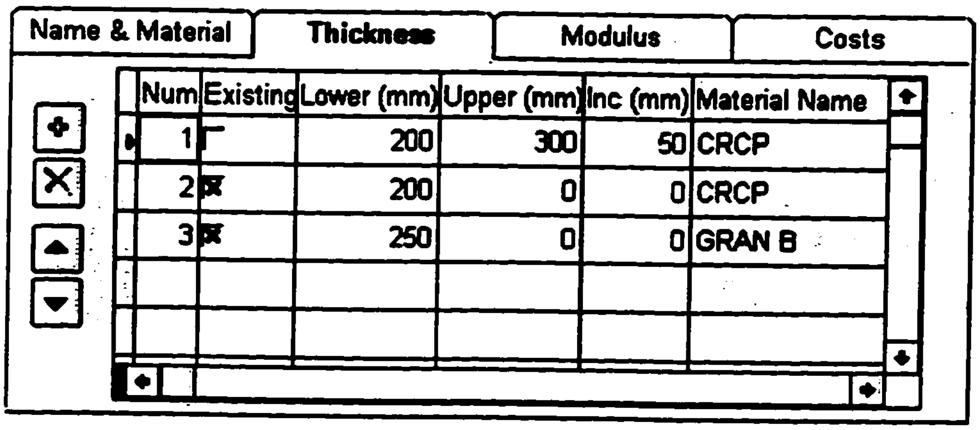
Material

Material names are selected from the Material Table in the Data Library. If a material to be used in the design does not appear on the dropdown list, the Material Table needs to be edited prior to inputting the pavement layer information.



Thickness

Use this field to define the thickness range for each layer.



Lower Boundary of Layer Depth

Defines the lower limit of thickness for the design layer. This value must be entered in millimetres.

Upper Boundary of Layer Depth

Defines the upper limit of thickness for the design layer. This value must be entered in millimetres.

Increment of Layer Depth

Determines the number of design alternatives based on the upper and lower boundaries of layer depth. The smaller the increment, the more design alternatives - this should be considered during design, as it may affect computing time.

Thickness of Existing PCC Slab (D)

The thickness (in mm) of the existing PCC slab should be obtained through review of original design from construction documents. Coring of the existing pavement is recommended to ensure the accuracy of this input.

Modulus

Name & Material | Thickness | **Modulus** | Costs

Num	Existing	Rupture	Elasticity	Load Transf	Material Name
1					CRCP
2	X	3.00	40000	4.00	CRCP
3	X		175		GRAN B

Modulus of Rupture

AASHTO suggests the 28-day mean value Sc' be calculated as:

$Sc'(mean) = Sc + z(SD_s)$

where,

Sc is construction specification on concrete modulus of rupture (MPa)

SDs = estimated standard deviation of concrete modulus of rupture (MPa)

z = standard normal deviate

Please consult the Engineering Document for more details.

Modulus of Elasticity (Ec)

The modulus of elasticity (Ec) of the concrete can be approximated as:

$Ec = 6.750 * Sc'$ (MPa)

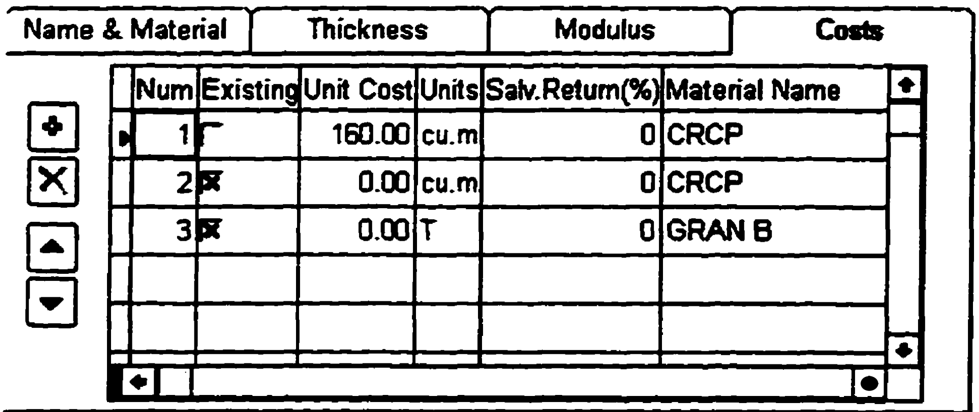
Load Transfer Coefficient (J)

Recommended Load Transfer Coefficient

Shoulder	Asphalt		Tied PCC	
Load Transfer Devices	Yes	No	Yes	No
Pavement Type				
Plain jointed and jointed reinforced	3.2	3.8-4.4	2.5-3.1	3.6-4.2
CRCP	2.9-3.2	N/A	2.3-2.9	N/A

Costs

Unit cost of materials. It can be in $/cu.m or $/Tonne.



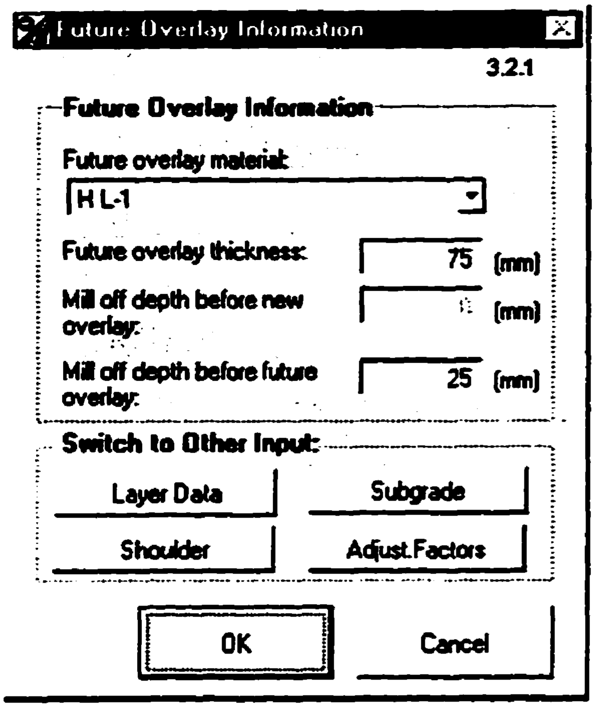
Num	Existing	Unit Cost	Units	Salv.Return(%)	Material Name
1		160.00	cu.m	0	CRCP
2		0.00	cu.m	0	CRCP
3		0.00	T	0	GRAN B

Dummy Layer

Allows using a layer that is only counted for the cost.

Future Overlay Data

The future overlay data input screen is shown below.

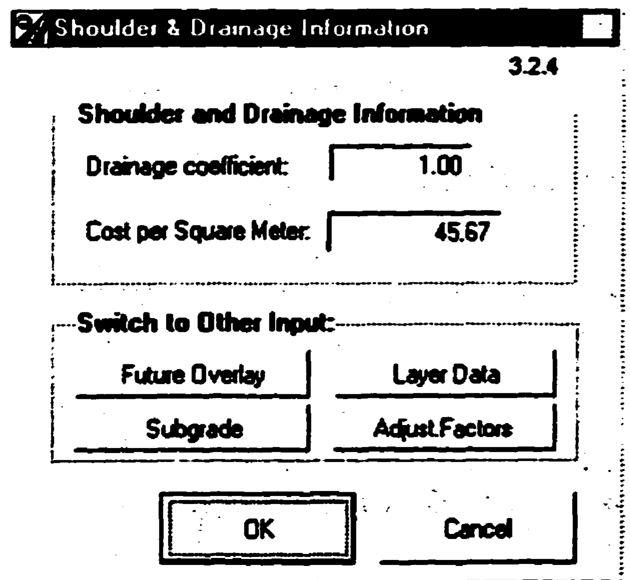


Future Overlay Thickness (PCC)

Defines the depth of future overlays which may be applied, if necessary, beyond the initial life of the pavement structure.

Shoulder and Drainage Data

The shoulder and drainage data input screen is shown below.



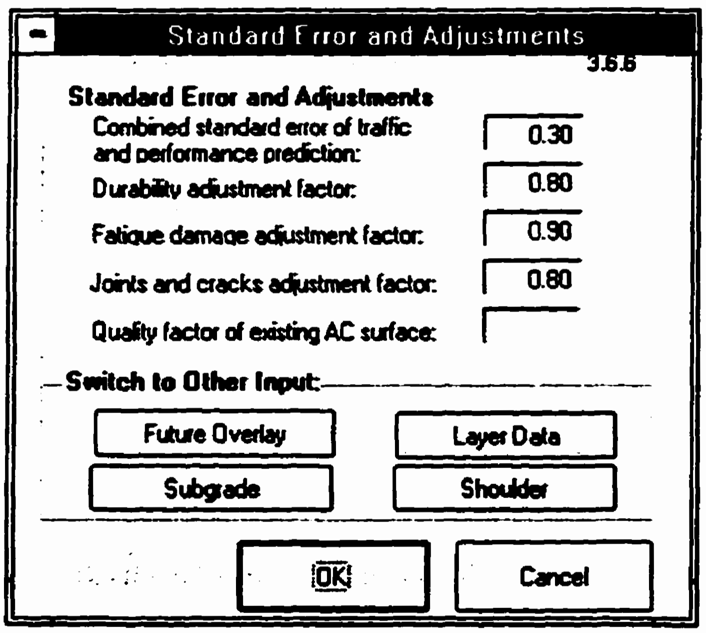
Drainage Coefficient (Cd)

Recommended Drainage Coefficient, C_d

Quality of Drainage	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
	<1%	1-5%	5-25%	>25%
Excellent	1.25-1.20	1.20-1.15	1.15-1.10	1.10
Good	1.20-1.15	1.15-1.10	1.10-1.00	1.00
Fair	1.15-1.10	1.10-1.00	1.00-0.90	0.90
Poor	1.10-1.00	1.00-0.90	0.90-0.80	0.80
Very Poor	1.00-0.90	0.90-0.80	0.80-0.70	0.70

Standard Error and Adjustment Factors (PCC)

The Standard Error and Adjustment Factors input screen is shown below.



Combined Standard Error (S_0)

The combined standard error of the traffic and performance prediction is a coefficient used in deriving the AASHTO design equations. A range of 0.3 - 0.4 is recommended by AASHTO for rigid pavements.

Joints and Cracks Adjustment Factor (Fjc)

According to Figure 5.12 in the AASHTO guide, this factor can be determined based on the condition survey of the existing PCC pavement. Recommended value 1.0, repair all deteriorated areas.

- 1.0-0.84: 0-40 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km

- 0.84-0.70: 41-80 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km
- 0.70-0.56: 81-120 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km
- 0.56: > 120 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km

Durability Adjustment Factor (Fdur)

This factor is determined based on the condition survey of the existing PCC pavement:

- 1.00: No sign of PCC durability problems
- 0.96-0.99: Some durability cracking exist, but no spalling
- 0.88-0.95: Substantial cracking and some spalling exist
- 0.80-0.88: Extensive cracking and severe spalling exist

Fatigue Damage Adjustment Factor (Ffat)

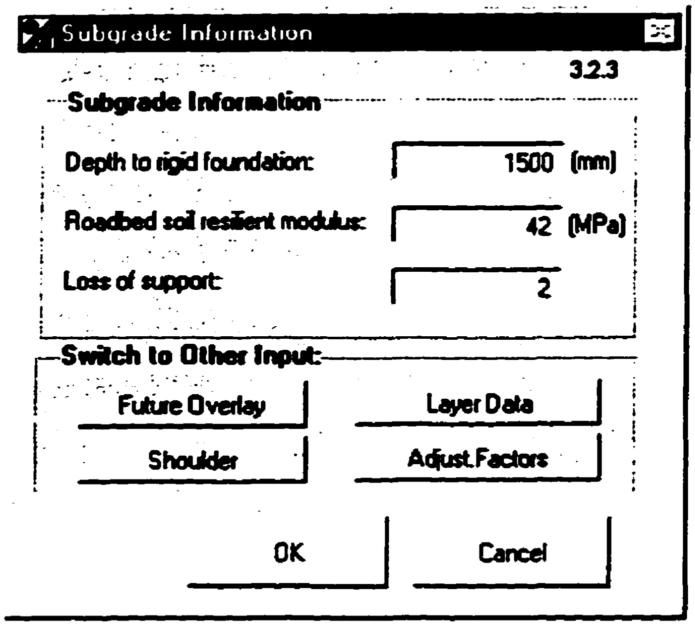
This factor is determined based on the condition survey of the existing PCC pavement:

- 0.97-1.00: Few transverse cracks/punchouts exist
  JPCP: < 5 percent slabs are cracked
  JRCP: < 16 working cracks per km
  CRCP: < 3 punchouts per km
- 0.94-0.96: A significant number of transverse cracks/punchouts exist
  JPCP: 5-15 percent slabs are cracked
  JRCP: 16-47 working cracks per km
  CRCP: 3-7 punchouts per km
- 0.90-0.93: A large number of transverse cracks/punchouts exist
  JPCP: > 15 percent slabs are cracked
  JRCP: > 47 working cracks per km
  CRCP: > 7 punchouts per km

Unbonded PCC Overlay of PCC Pavement

Subgrade Data (PCC)

The subgrade data input screen is shown below.


Subgrade Information
3.2.3
Subgrade Information
Depth to rigid foundation:
1500
(mm)
Roadbed soil resilient modulus:
42
(MPa)
Loss of support:
2
Switch to Other Input:
Future Overlay
Layer Data
Shoulder
Adjust Factors
OK
Cancel

Roadbed Soil Resilient Modulus (M_R)

AASHTO Guide suggests using a relationship of 1500*CBR for the subgrade resilient modulus, Mr (psi). Typical value: 7-345 Mpa.

Loss of Support (LS)

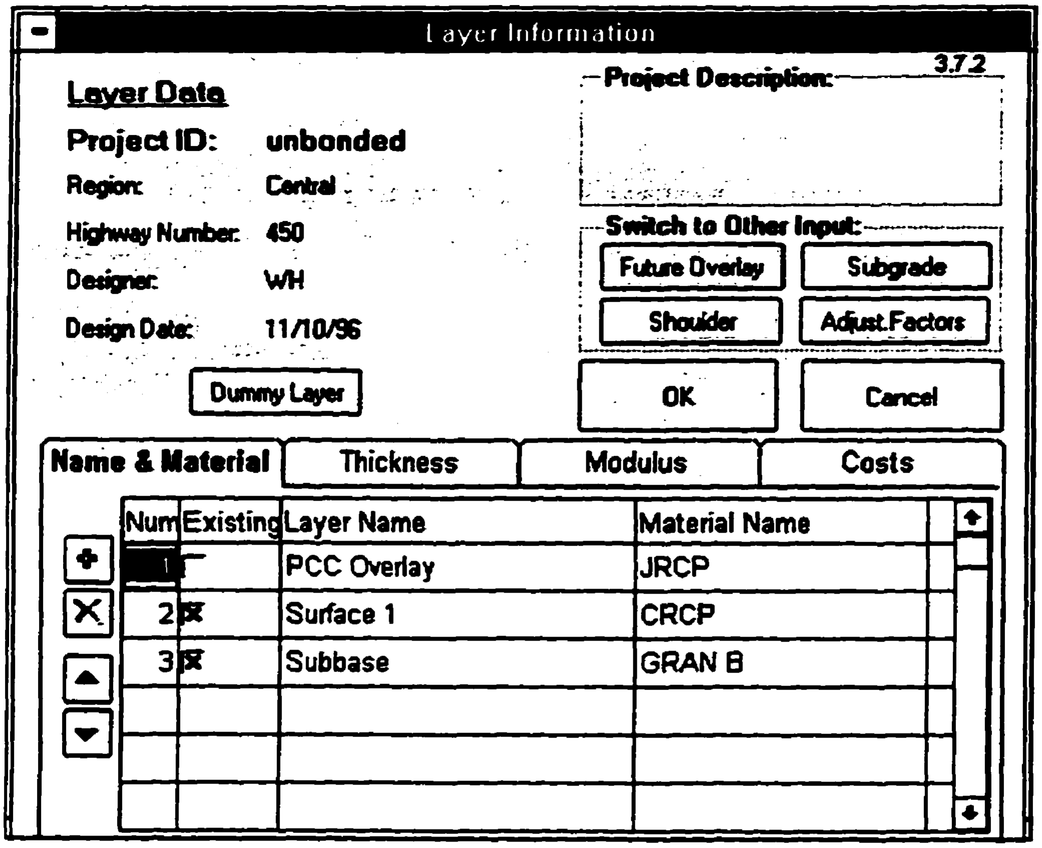
Typical LS values:

Type of material	LS
Cement treated granular base (E=7000-14000 MPa)	0 to 1
Cement aggregate mixtures (E=3450-7000 MPa)	0 to 1
Asphalt treated base (E=2400-7000 MPa)	0 to 1
Bituminous stabilized mixtures (E=280-2100 MPa)	0 to 1
Lime stabilized (E=140-480 MPa)	1 to 3
Unbound granular materials (E=100-310 MPa)	1 to 3
Fine grained or natural subgrade materials (E=20-280 MPa)	2 to 3

Layer Data (PCC)

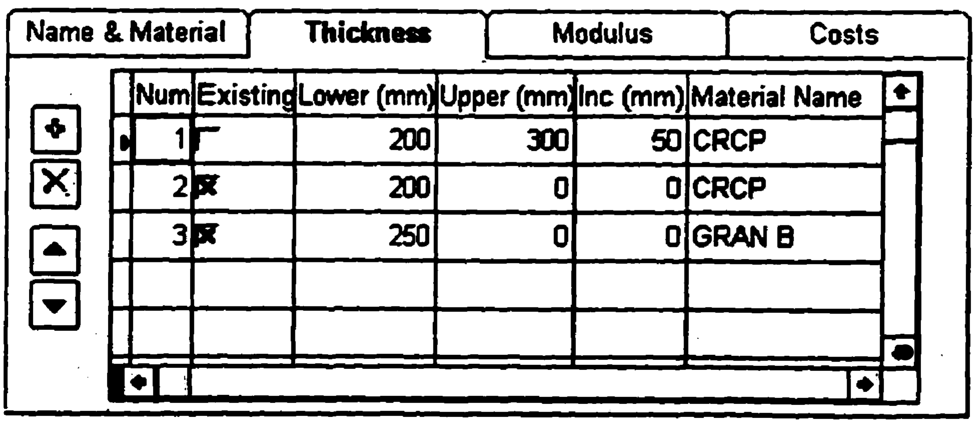
Material

Material names are selected from the Material Table in the Data Library. If a material to be used in the design does not appear on the dropdown list, the Material Table needs to be edited prior to inputting the pavement layer information.



Thickness

Use this field to define the thickness range for each layer.



Lower Boundary of Layer Depth

Defines the lower limit of thickness for the design layer. This value must be entered in millimetres.

Upper Boundary of Layer Depth

Defines the upper limit of thickness for the design layer. This value must be entered in millimetres.

Increment of Layer Depth

Determines the number of design alternatives based on the upper and lower boundaries of layer depth. The smaller the increment, the more design alternatives - this should be considered during design, as it may affect computing time.

Thickness of Existing PCC Slab (D)

The thickness (in mm) of the existing PCC slab should be obtained through review of original design from construction documents. Coring of the existing pavement is recommended to ensure the accuracy of this input.

Modulus

Name & Material | Thickness | **Modulus** | Costs

Num	Existing	Rupture	Elasticity	Load Transf	Material Name
1					CRCP
2	X	3.00	40000	4.00	CRCP
3	X		175		GRAN B

Modulus of Rupture

AASHTO suggests the 28-day mean value Sc' be calculated as:

$Sc'(mean) = Sc + z(SD_s)$

where,

Sc is construction specification on concrete modulus of rupture (MPa)

SDs = estimated standard deviation of concrete modulus of rupture (MPa)

z = standard normal deviate

Please consult the Engineering Document for more details.

Modulus of Elasticity (Ec)

The modulus of elasticity (Ec) of the concrete can be approximated as:

$Ec = 6.750 \cdot Sc'$ (MPa)

Load Transfer Coefficient (J)

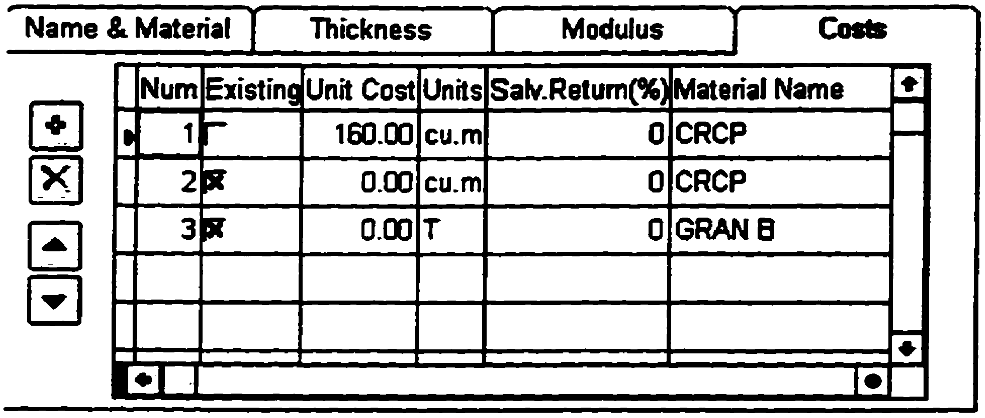
Recommended Load Transfer Coefficient

Shoulder	Asphalt		Tied PCC	
Load Transfer Devices	Yes	No	Yes	No
Pavement Type				

Plain jointed and jointed reinforced	3.2	3.8-4.4	2.5-3.1	3.6-4.2
CRCP	2.9-3.2	N/A	2.3-2.9	N/A

Costs

Unit cost of materials. It can be in $/cu.m or $/Tonne.



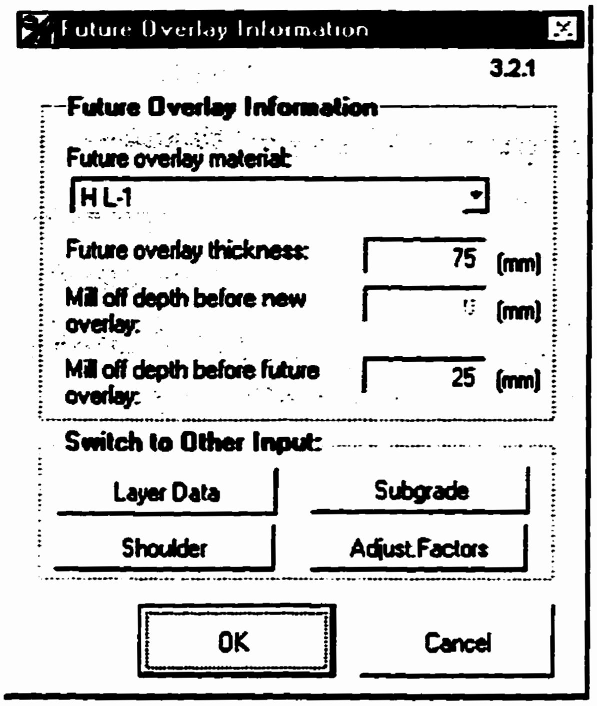
Num	Existing	Unit Cost	Units	Salv.Return(%)	Material Name
1		160.00	cu.m	0	CRCP
2		0.00	cu.m	0	CRCP
3		0.00	T	0	GRAN B

Dummy Layer

Allows using a layer that is only counted for the cost.

Future Overlay Data

The future overlay data input screen is shown below.

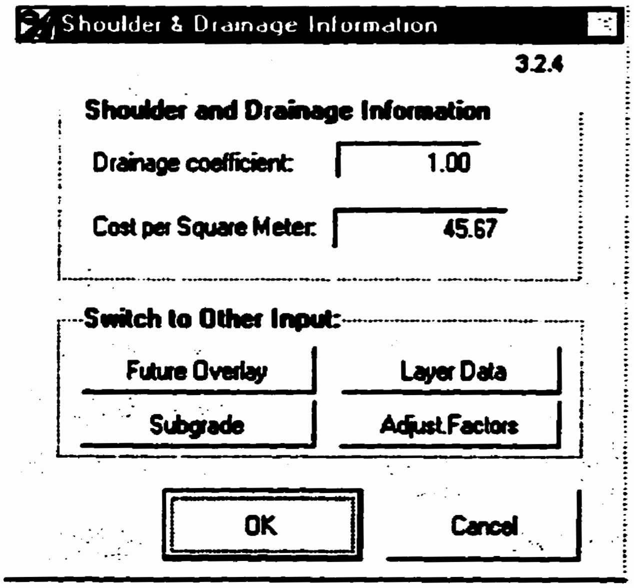


Future Overlay Thickness (PCC)

Defines the depth of future overlays which may be applied, if necessary, beyond the initial life of the pavement structure.

Shoulder and Drainage Data

The shoulder and drainage data input screen is shown below.



Drainage Coefficient (Cd)

Recommended Drainage Coefficient, C_d

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	<1%	1-5%	5-25%	>25%
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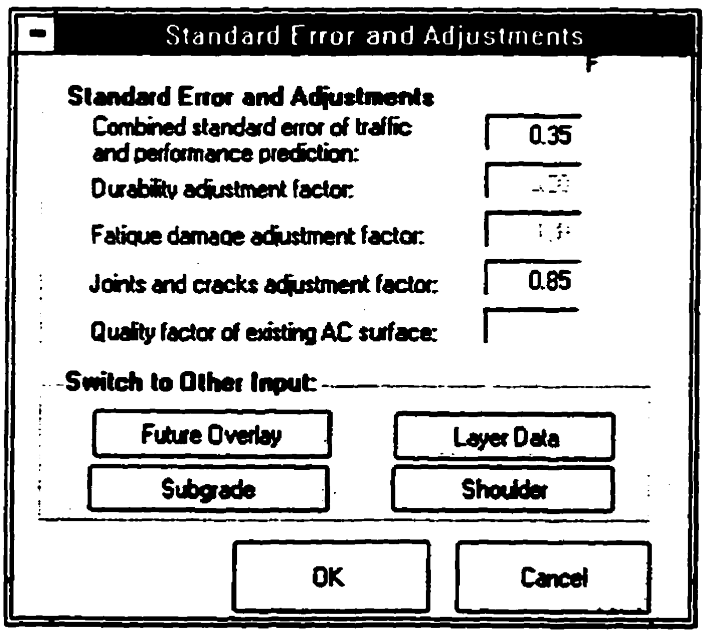
Good	1.20-1.15	1.15-1.10	1.10-1.00	1.00
Fair	1.15-1.10	1.10-1.00	1.00-0.90	0.90
Poor	1.10-1.00	1.00-0.90	0.90-0.80	0.80
Very Poor	1.00-0.90	0.90-0.80	0.80-0.70	0.70

Standard Error and Adjustment Factors (PCC)

The Standard Error and Adjustment Factors input screen is shown below.



Combined Standard Error (S_0)

The combined standard error of the traffic and performance prediction is a coefficient used in deriving the AASHTO design equations. A range of 0.3 - 0.4 is recommended by AASHTO for rigid pavements.

Joints and Cracks Adjustment Factor (Fjc)

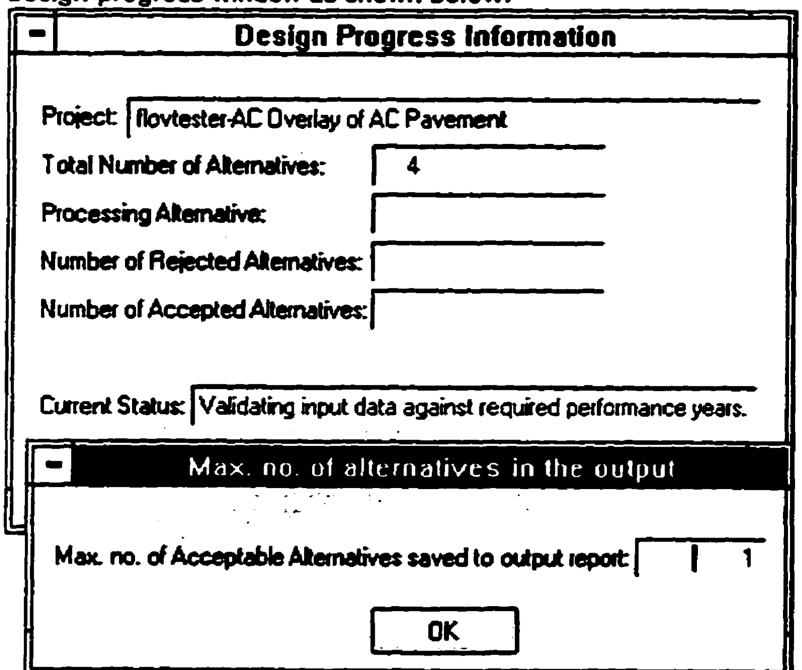
According to Figure 5.12 in the AASHTO guide, this factor can be determined based on the condition survey of the existing PCC pavement. Recommended value 1.0, repair all deteriorated areas.

- 1.0-0.84: 0-40 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km

- 0.84-0.70: 41-80 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km
- 0.70-0.56: 81-120 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km
- 0.56: > 120 unrepaired deteriorated joints, cracks, punchouts and wide expansion joints/km

Design Alternatives

To begin processing the data which has been input for the design alternative, select 'Design Alternatives' from the Design menu. The program will examine the design input and bring up a design progress window as shown below:



After selecting the number of design alternatives to be processed, OPAC 2000 will start the structural and economic analysis. The results will be saved in the on-line database and they can be viewed and printed from the report menu.

FWD Backcalculation

OPAC 2000 can be used for rigid and composite pavement backcalculations based on falling weight deflectometer (FWD) data which has been input into the system. The output of the backcalculation includes the PCC rupture modulus, the elastic modulus, the subgrade reaction as well as the load transfer coefficient J. These results can be used for design inputs of rigid and composite pavement overlays. To use the backcalculation tool select 'Backcalculation' from the Design menu.

5.0 Reporting

Their are 7 different reports available from the Reporting menu. These are:

Input Report
Design Alternatives Report

Performance Curves
Sensitivity Report
Cross-Section Report
Emissions Report

Input Report

The input report includes all input data including Section Information , Traffic Information , Economic Information , and Design Information and Analysis . An on-screen preview is available as well as a hard copy report.

OPAC2000 - Project Data Report

New Flexible (AC) Pavement Design

Project ID: testlanes

Project Description

Highway Number: 111
Region: Southwest
District: District 3
Design Date: 1996/05/21
Designer:

Description:
Offset: 1.00 km, Length: 10.40 km

Section Identification

LHRS: 22222
No. of Lanes: 6
Lane Width: 3.75 metres

Direction: Both
Shoulder Width:
Outer: 3.00 metres
Inner: 1.50 metres
Divided/Undivided: Divided

Design Alternatives Report

The design alternatives report summarizes the results of the design analysis. This report shows the project costs, the layer thicknesses, the initial life and overlay life, and the equivalent thicknesses for each design alternative. Also, the Project ID, Highway Number, Linear Highway Referencing System (LHRS) Number, Offset, Design Date, and Designer are all shown.

A maximum of 8 alternatives can be displayed on each page. An on-screen preview is available as well as a hard copy report.

OPAC2000
Ministry of Transportation of Ontario

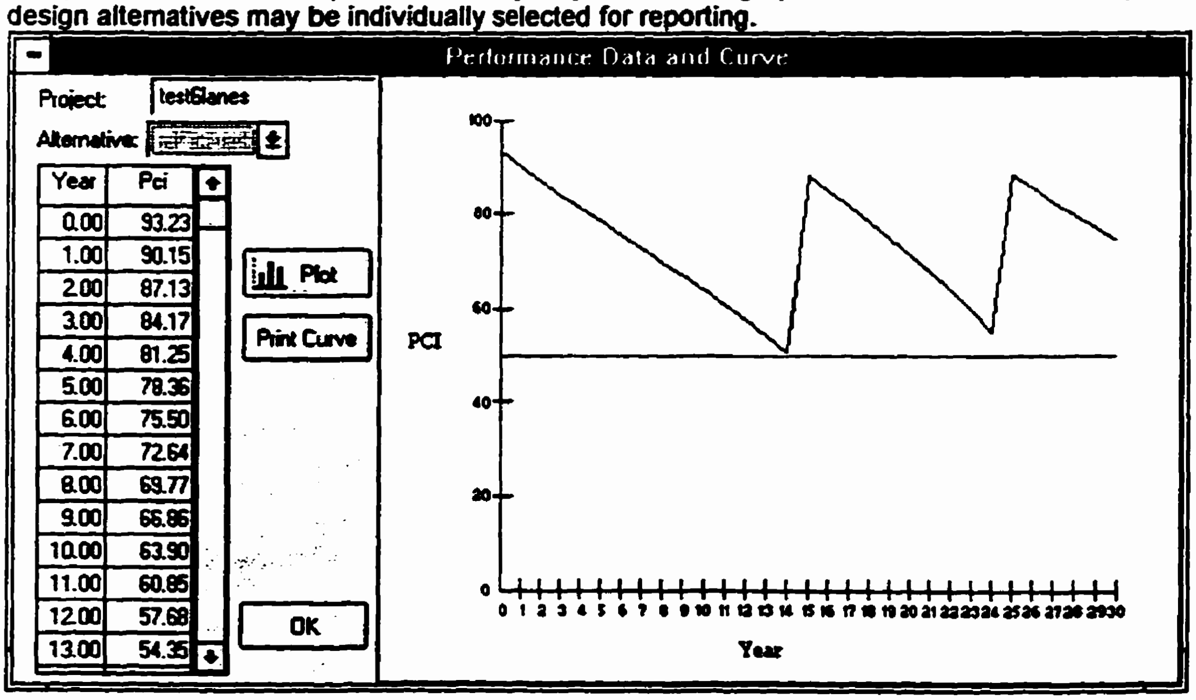
New Flexible (AC) Pavement

Project ID: test6lanes
Highway # : 111
LHRS # : 22222
Offset : 1.00
Design Date: 1996/05/21
Designer :

Alternative	1	3	2
Cost ($/km)			
Initial Const.	727515	748215	753390
Rehabilitation	126550	120524	118341
UDL Cost	13935	15847	17760
Maint.Cost	117613	119492	120184
Residual Value	-49857	-53164	-55018
Total Cost	935756	950914	954657
Layer Depth (mm)			
D(1)	40	40	50
D(2)	110	120	110
D(3)	140	140	140
D(4)	375	375	375

Performance Curves

The performance curve report shows the yearly PCI data in graphic form. Each of the accepted design alternatives may be individually selected for reporting.



Sensitivity Report

A sensitivity analysis may be performed on the design results based on 4 possible independent variables; minimum acceptable PCI, growth rate, reliability, and AADT. A cost type to be displayed on the graph may also be selected, along with the number of points to display. Be aware that increasing the number of points may increase the processing time.

View Layer

Displays the layer thickness of the section for the selected alternative.

Independent Variable

Selected an independent variable for performing sensitivity analysis. The variable will be analyzed through the range identified below.

Cost Type

Identifies which type of cost to display: total cost, user cost, or agency cost.

View Data

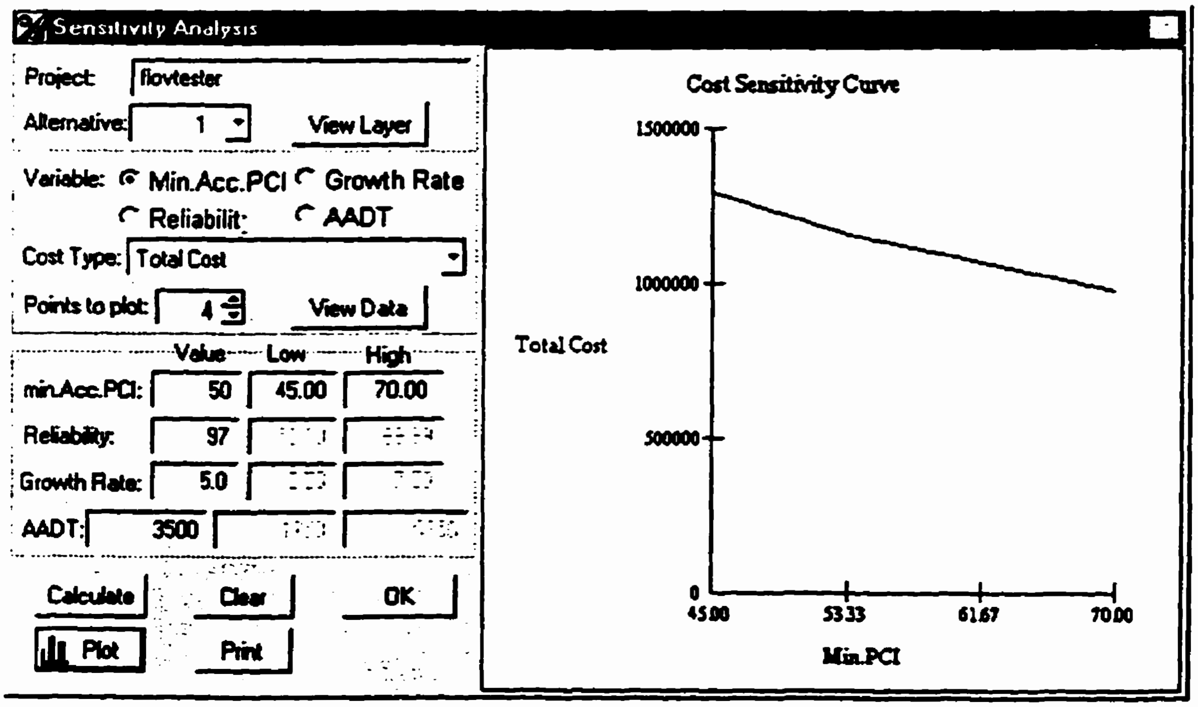
Displays the calculated data of the dependent variable, based on the identified range of the independent variable.

Variable Range for Sensitivity Analysis

Identifies the range over which the independent variable will be analyzed.

Plot

Plots the selected cost type based on the independent variable range.

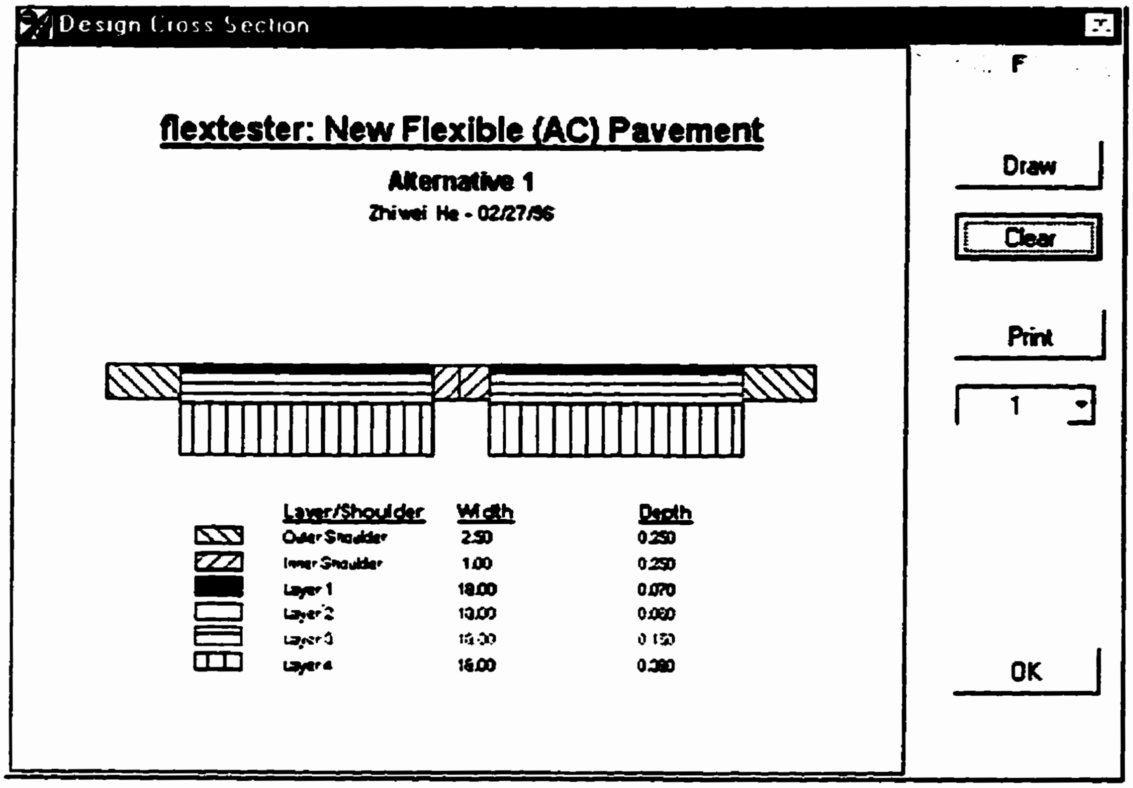


Calculate

Calculates the sensitivity analysis results given the identified inputs.

Cross-Section Report

The Cross-Section report shows a layer by layer section of the design roadway, based on a selected alternative. An example of this report is shown below:



Emissions Report

The emissions report gives the predicted amount of CO and HC associated with a design alternative selected by the user.

6.0 Glossary

-A-

AADT

Average annual daily traffic during the design year.

Analysis Period

The period through which the pavement design will be analyzed. 30 years is suggested for flexible pavement designs, 40 years for rigid pavement designs.

-B-

Base Year Maintenance Cost

Expected cost of maintenance during the design year.

-C-

C.O.V. of GBE

Coefficient of Variance of the granular base equivalency factor (GBE).

C.O.V. of Subgrade Strength

Coefficient of Variance of subgrade strength.

C.O.V. of Traffic Estimation

Coefficient of Variance of traffic estimation.

Combined Standard Error (S_0)

Combined standard error of the traffic and performance prediction.

-D-

Directional Split Factor

This factor is used for converting two-way traffic into one-way traffic.

Discount Rate

Compound rate used for calculating the present worth of future costs. It represents a blend between expected rate of return and expected rate of inflation.

Dummy Layer

Allows using a layer that is only counted for the cost.

Durability Adjustment Factor (Fdur)

This factor is determined based on the condition survey of the existing PCC pavement:

-F-

Fatigue Damage Adjustment Factor (Ffat)

This factor is determined based on the condition survey of the existing PCC pavement:

FHWA Vehicle Classification

Federal Highway Administration (U.S.) vehicle classification breaks vehicles into 13 different classes. The last 10 of these classes are truck classifications.

Future Overlay Depth

Depth of future AC or PCC overlays which may be applied, if necessary, beyond the initial life of the pavement structure.

-G-

Growth Rate (GR)

Annual traffic growth rate. The growth rate can be linear or geometric (similar to compound interest).

GBE

Granular Base Equivalency factors

-I-

Increment of Layer Depth
Used to determine the number of design alternatives based on the upper and lower boundaries of layer depth. The smaller the increment, the more design alternatives.

Initial Life
Minimum required life of pavement to the first overlay.

-J-

Joints and Cracks Adjustment Factor (Fjc)
According to Figure 5.12 in the AASHTO guide, this factor can be determined based on the condition survey of the existing PCC pavement. Recommended value 1.0, repair all deteriorated areas.

-L-

Lane Distribution Factor (LDF)
Distributes traffic to the design lane according to the AADT and the number of lanes.

Lower Boundary of Layer Depth
Defines the lower limit of thickness for the design layer. This value must be entered in millimetres.

-M-

Maintenance Cost Increase
The expected increase in maintenance costs each year. It can be identified as a fixed increment or as a percent increase (similar to compound interest).

Maintenance Schedule
The maintenance schedule provides the user with an opportunity to identify particular maintenance treatments which are anticipated during a particular year of the roadway section's life.

-P-

Poisson's Ratio
Poisson's ratio ranges from 0.1 to 0.4 for pavement materials. The typical value for a concrete slab is 0.15.

-R-

Roadbed Soil Resilient Modulus (M_R)
AASHTO Guide suggests using a relationship of 1500*CBR for the subgrade resilient modulus, M_R (psi). Typical value: 7-345 MPa.

-S-

Simplified Truck Classification
A simplification of the FHWA vehicle classification which divides truck traffic into 4 vehicle classes.

Subgrade Condition
The subgrade condition is used to determine the strength of the subgrade.

Subgrade Type
The subgrade type is used to determine the strength of the subgrade.

-T-

Thickness of Existing PCC Slab (D)
The thickness of the existing PCC slab should be obtained through review of original design and construction documents. Coring of the existing pavement is recommended to ensure the accuracy of this input.

Truck Percent (T%) and Truck Factor (TF)
There are 2 vehicle classification schemes available. These are the FHWA vehicle classification and the simplified truck classification. In both cases the truck traffic is divided

into a number of different classifications and a Truck Factor (TF) assigned to each. A percent distribution must be entered to indicate what share of the total truck traffic each class represents.

-U-

Upper Boundary of Layer Depth

Defines the upper limit of thickness for the design layer. This value must be entered in millimetres.